1915
B14

BADE

Hydro-Electric Project on the Ottertail River, Minnesota

Electrical Engineering

M. S.

1915

UNIV. OF
ILLINOIS
LIBRARY

THE UNIVERSITY
OF ILLINOIS
LIBRARY
1915
B14

HYDRO-ELECTRIC PROJECT
ON THE
OTTERTAIL RIVER, MINNESOTA

BY

COURTLAND WALTER BADE
E.E. Technicum Mittweida, Germany, 1914

THESIS

Submitted in Partial Fulfillment of the Requirements for the

Degree of

MASTER OF SCIENCE

IN ELECTRICAL ENGINEERING

IN

THE GRADUATE SCHOOL

OF THE

UNIVERSITY OF ILLINOIS

1915

Digitized by the Internet Archive
in 2016

https://archive.org/details/hydroelectricpro00bade

1915
B14

UNIVERSITY OF ILLINOIS

THE GRADUATE SCHOOL

June 1 1915

I HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER MY SUPERVISION BY Courtland Walter Bade

ENTITLED Hydro-Electric Project on the Ottertail River, Minnesota

BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF Master of Science in Electrical Engineering

Edw. H. Waldo
In Charge of Thesis

Ellery B. Paine
Head of Department

Recommendation concurred in:*

Committee on Final Examination*

*Required for doctor's degree but not for master's.

1315
B14

CONTENTS

318139

I.

PRESENT POWER DEVELOPMENTS IN THE RED RIVER BASIN

Before the extensive development of water power was thought of, numerous towns and cities in the Red River Basin had installed steam-power plants for lighting purposes. Among these were Wahpeton, Graceville, Morris, Elbow Lake and Hankinson. With a head of 65 ft., the city of Fergus Falls has developed sufficient power for its own use for the last forty years, this being the oldest development in the Red River Basin. The Ottertail Power Company, which was incorporated in 1909 bought up the steam-power plants in the above mentioned cities, also the hydro-electric plant at Fergus Falls, and developed a new station on the Ottertail River, two miles above Fergus Falls. This new development is the Hoot Lake project, having a head of 70 feet and develops 2500 h. p. and is not a dam but a diversion proposition. Where the river reaches a point within three miles of the city it has to make a loop of about twelve miles to get thru the bluffs; it is here diverted and led to the top of the bluffs thru a few short canals and natural reservoirs giving it a drop of 70 ft.

In recent years the United States Geological Survey has surveyed the entire Red River Basin to determine the amount of water-power available. From Wahpeton to the Canadian boundary line the slope of the Red River is so small that a possibility of power development is quite out of the question as there are no natural reservoirs. The Red Lake River, the northern most tributary of any size in the United States, is a very reliable river in its flow and has two points very suitable for small power developments. This river flows in the most sparsely populated

districts of Minnesota, and it is questionable if at any time in the near future it would pay to develop power there. The only other river for power development is the Ottertail River on which five favorable sites are situated, all within a distance of ten miles of the city of Fergus Falls. To meet the increasing demand for electricity a third site is to be developed at a point seven miles below the entrance of the Pelican River. The two remaining sites are controlled by the Ottertail Power Company and altho of smaller capacity will be improved as soon as there is a demand for the power. The company controlling sites for hydro-electric developments within a radius of one hundred miles and also owning the present steam-power plants, it is practically subject to no competition whatever.

On the Pelican River and other smaller tributaries of the Red River a number of dams have been built and power for milling purposes developed, seldom exceeding 50 h. p. These are:

Place	Stream		Head	Av. h. p.
Maine Mills	Ottertail	River	8 ft.	50
Lakeview	Pelican	"		50
Kingsburg	"	"	11 ft.	30
Pelican Rapids	"	"	12 ft.	100
Elizabeth	"	"	13 ft.	100
Richwood	Buffalo	"	11 ft.	40
Fertile	Sand Hill	"	15 ft.	50

At Pelican Rapids and Elizabeth the power for lighting purposes is supplied by the mills.

II.

A. GENERAL FEATURES OF THE AREA DRAINED BY THE RED RIVER

The Red River rises in Minnesota, its most remote source being a small lake near the southwest corner of Clearwater County, about 13 miles from Lake Itasca, at an elevation of about 1550 ft. above sea level. From this lake it flows southward 60 miles through a succession of small lakes to Ottertail Lake (elev. 1320 ft.), thence to Breckenridge; from this point it runs northward 285 miles to Lake Winnipeg. Its slope rarely exceeds 6 inches per mile, and on account of the meandering of the river, the length of its channel is nearly double the length of the direct line of flow.

During the glacial epoch the upper part of the Red River Valley was occupied by an immense lake, now called Lake Agassiz, which had its outlet through the lower part of this valley and Big Stone Lake to the present Minnesota Valley. At present there is water connection between the two lakes during times of high water only, as the watershed between the two is a marsh that is only three ft. above Lake Traverse and 11 ft. above Big Stone Lake. The valley of the Bois de Sioux, which extends from Big Stone Lake to Lake Winnipeg is a plain from 30 to 50 miles wide and 315 miles long. Lake Traverse is 15 miles long and from 1 to 1 1/2 miles wide and is shallow, being for the most part less than 10 ft. deep; it is bordered on either side by bluffs from 100 to 150 ft. high which continue down a narrow valley to Big Stone Lake. The slope of the valley to the Canadian boundary averages 0.46 ft. per mile.

B. TRIBUTARIES OF THE RED RIVER

a. The Ottertail River

Of the two streams which unite to form the Red River, the Ottertail furnishes 90 % of the stream flow. From its source, a small lake on the west slope of the Leaf Hills, it flows southward passing through or uniting with a great number of lakes scattered over Becker and Ottertail Counties, the principal of which are: Height of Lared, Pine, Rush, Battle, East Battle and Ottertail Lakes. After its confluence with its chief tributary, Pelican River, it flows in a westerly direction. From Ottertail Lake to Breckenridge its fall is 385 ft., nearly all occurring above or near Fergus Falls.

b. The Pelican River

The Pelican River rises in Rice Lake, Becker County, and flows southward through a chain of lakes, the chief of which are: Floyd, Elsa, Detroit, Sallie Melissa, Lizzie and Lida Lakes.

C. TOPOGRAPHY OF THE OTTERTAIL RIVER DRAINAGE AREA.

The Ottertail River lies in that region of many lakes known as the Park Region of Minnesota. In Ottertail County alone there are over 1000 lakes, the largest being Ottertail Lake itself which is 8 miles long and 2 1/2 miles wide. Below the outlet of Ottertail the river passes through a slightly wooded rolling prairie. The lake region itself is heavily timbered and decidedly hilly.

D. GEOLOGY

The older rock formations throughout the greater part of the basin are covered with cretaceous rocks upon which rests

Transmission System.

Fig 1.

- Indi[illegible] St[illegible] Res[illegible] Town.
- " [illegible]
- " [illegible]
- Villages.

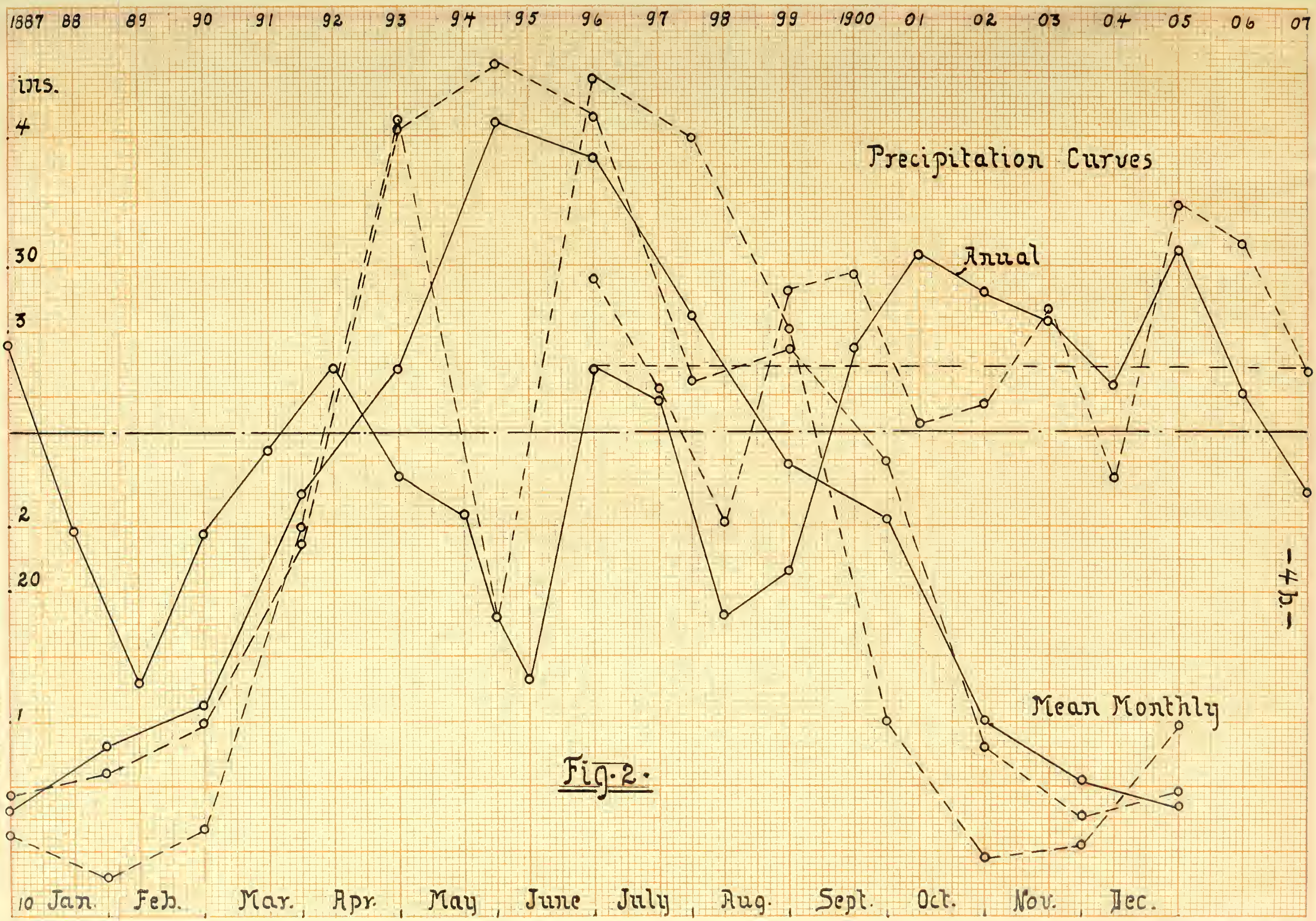
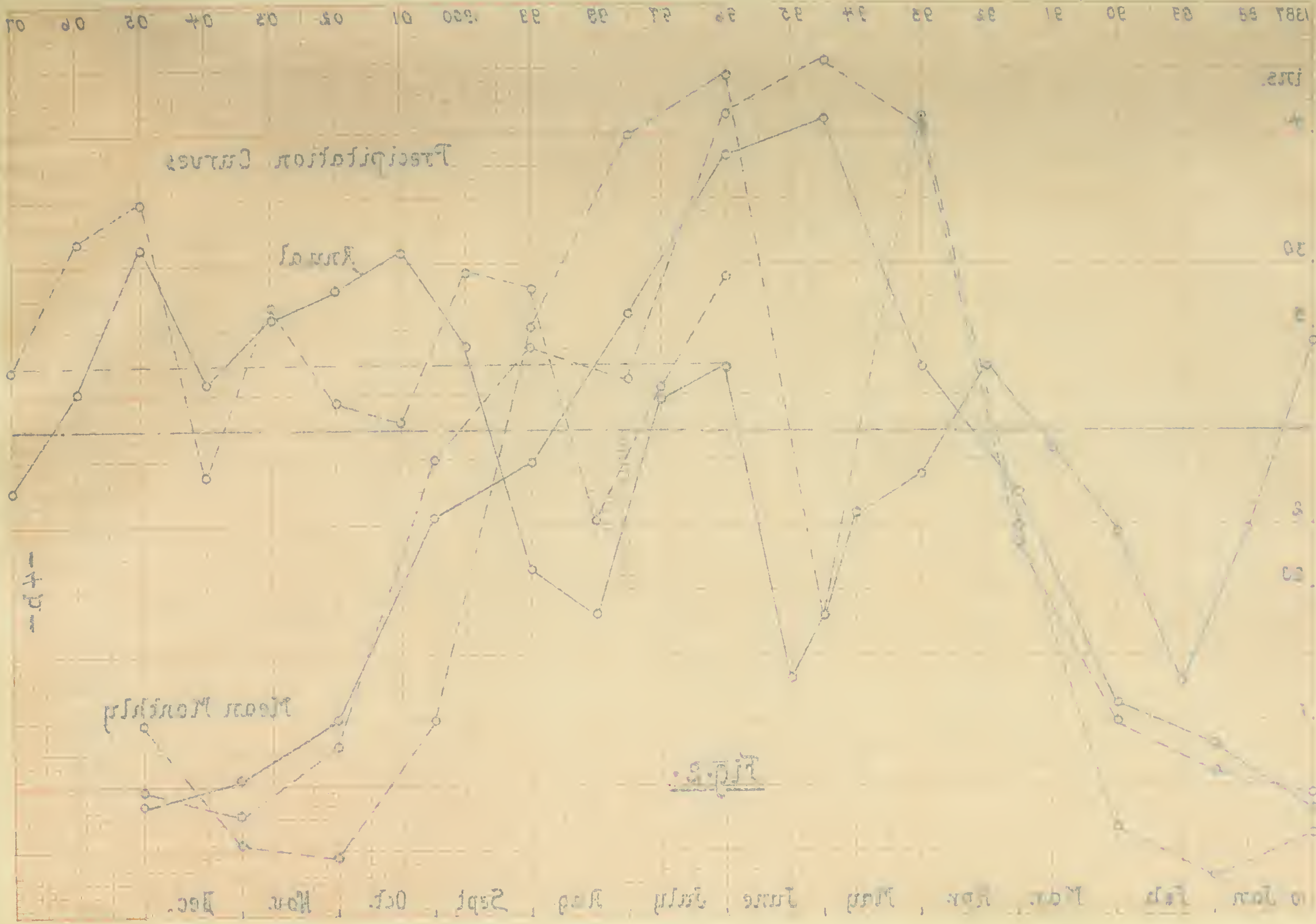

Precipitation Curves
Annual
Mean Monthly
ins.
4
30
3
2
20
1
10
1887
88
89
90
91
92
93
94
95
96
97
98
99
1900
01
02
03
04
05
06
07
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

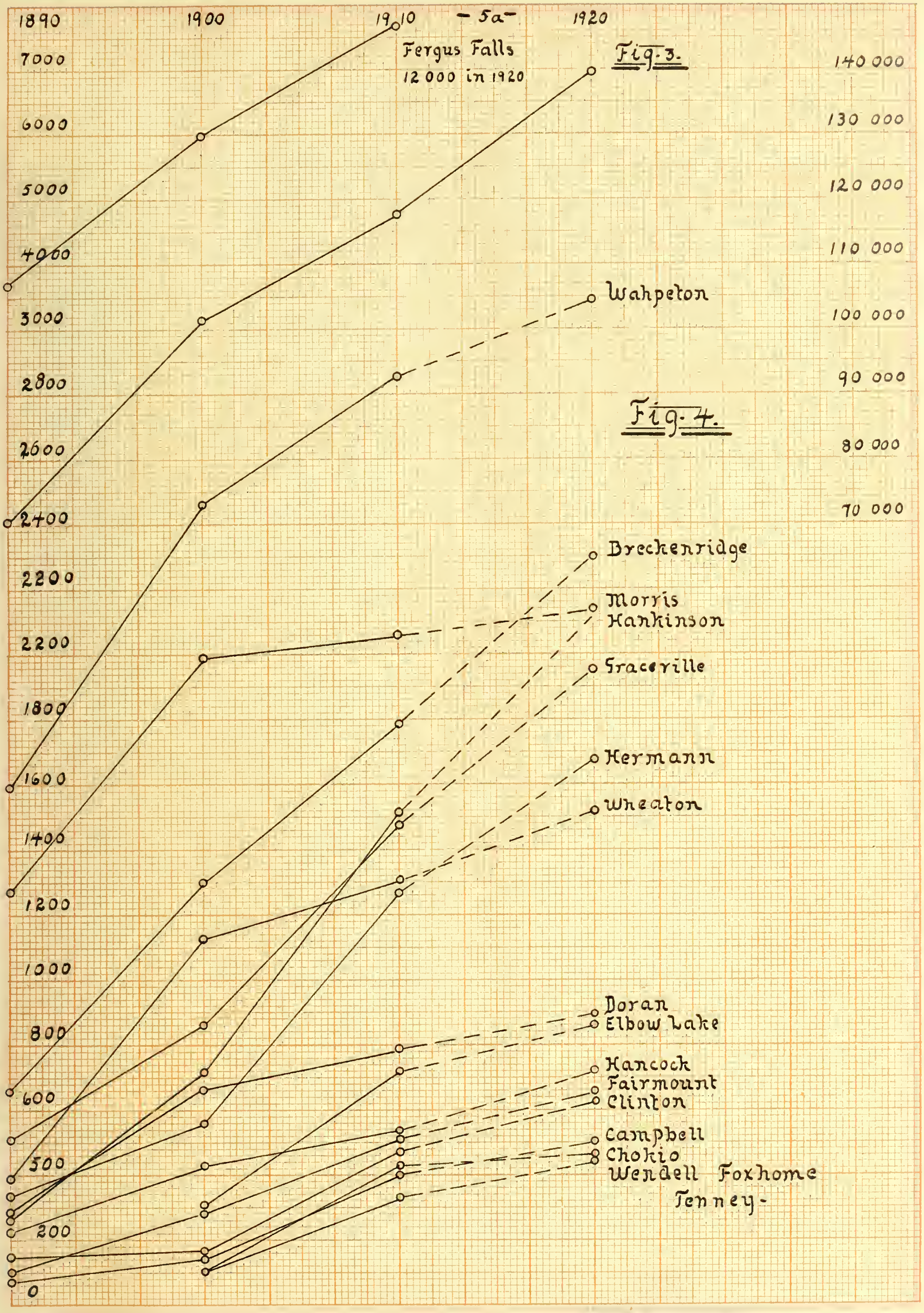
Fig. 2.

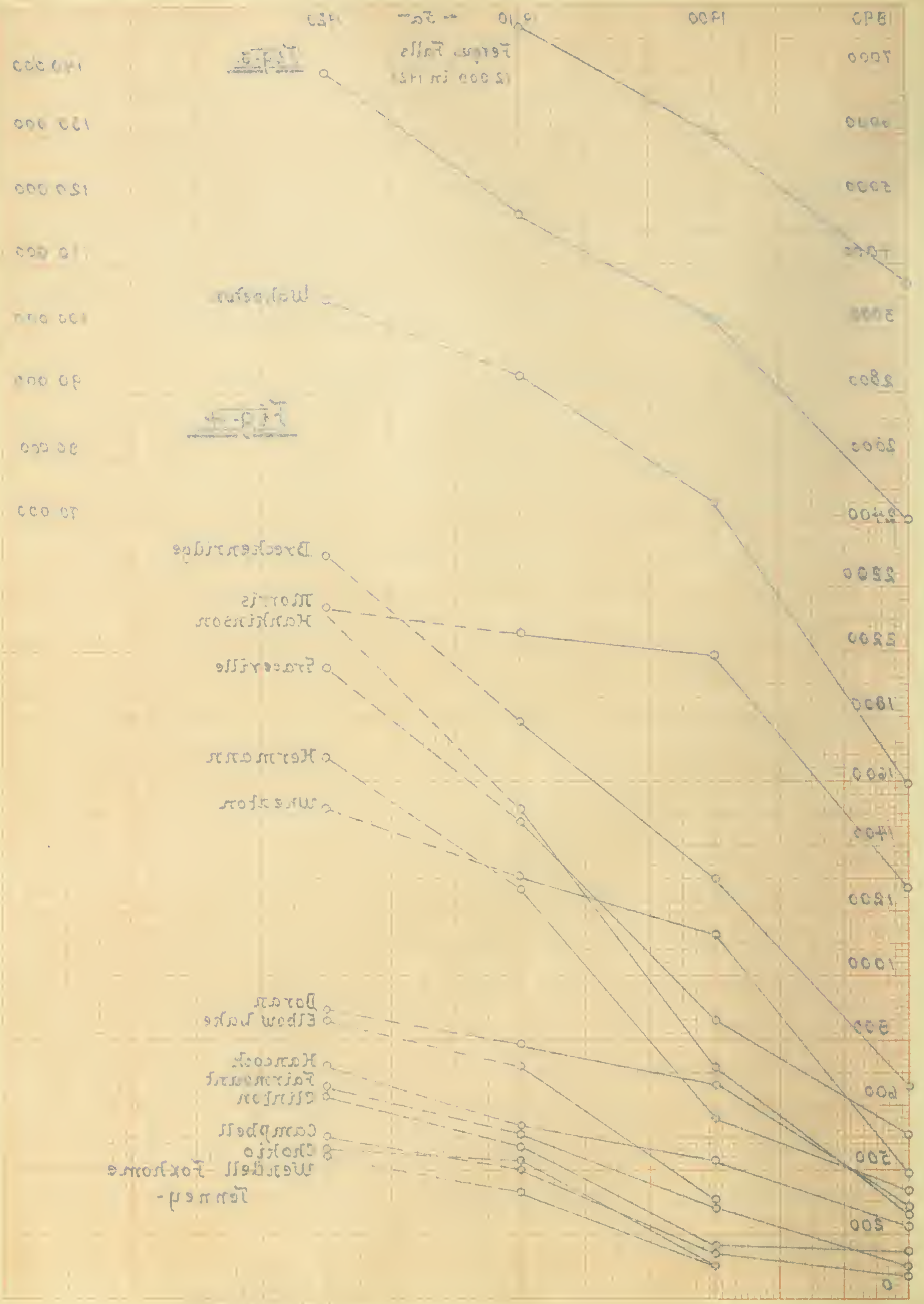


a heavy mantel of Pleistocene deposits. The earliest deposits consist of white and red sandstone and yellow clay and furnish strong artesian flow in many parts. That portion of the basin formerly occupied by Lake Agassiz is covered with a deposit of lacustrine clay; the entire area is covered with a sheet of blue till, consisting of a mixture of sand, clay and gravel.

E. PRECIPITATION IN THE RED RIVER BASIN

The mean annual rainfall in the Red River Drainage area increases uniformly from west to east, being 15 to 18 inches at the western boundary, 19 to 26 in. in the middle of the valley, and 24 to 30 in. at the eastern boundary. Owing to the larger rainfall on the eastern side of the area, the run-off per square mile from the tributaries on this side is from 2 to 10 times that on the west side depending on the topography of the district. The best reservoir site in the portion of the basin drained by the principal river itself is Ottertail Lake which has an area of about 22 sq. mi. and receives the run-off from an area of 1,160 sq. mi. Fig. 1 gives a map of the entire drainage basin and Fig. 2 shows the mean yearly and mean monthly precipitation for 30 years. From this curve it can be seen that the annual precipitation has dropped below 20 in. only twice in twenty years, and that the mean yearly precipitation for 20 years was 24.9 in. The dotted line gives the mean annual rainfall for Becker, Ottertail and Detroit Counties for a period of twelve years, giving an average of 26.92 in. From April until October 75% of the annual precipitation is received, making the distribution of precipitation for the year a fairly uniform one.


1890
1900
1910
1920
Fergus Falls
12 000 in 1920
Fig. 3.
Fig. 4.
7000
6000
5000
4000
3000
2800
2600
2400
2200
2200
1800
1600
1400
1200
1000
800
600
500
200
0
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
Wahpeton
Breckenridge
Morris
Hankinson
Graceville
Hermann
Wheaton
Doran
Elbow Lake
Hancock
Fairmount
Clinton
Campbell
Chokio
Wendell
Foxhome
Tenney



F. POPULATION

Since the Red River basin is devoted principally to wheat raising and lumbering, it is thinly populated and the size of its towns or cities is determined by the necessity of trading centers and not by the development of manufacturing interests. The population is less than half as dense as the average for the state as a whole and altho statistics show that the population is increasing, it is doubtful whether the Red River basin will ever be as thickly populated as the rest of the state where manufacturing interests are more predominant. Fig. 3 gives a curve for the increase of population in the entire basin and Fig. 4 shows the increase for the towns and cities supplied with power. The numbers indicate the curves for the following:

1	Fergus Falls	9	Chokio
2	Breckenridge	10	Clinton
3	Campbell	11	Doran
4	Elbow Lake	12	Fairmount
5	Hermann	13	Graceville
6	Foxhome	14	Hankinson
7	Wheaton	15	Johnson
8	Wahpeton	16	Morris
		17	Tenney
		18	Wendell
		19	Hancock

G. INDUSTRIES

Besides a few mills, grain elevators, railroad repair shops and saw-mills, few industries have been developed. The mills are usually situated in some small village on the river having their own water power and supplying the village with light.

Other mills are run during the summer and fall and are shut down during the winter months. The average h. p. of these seldom exceeds 50.

The Red River Basin is traversed by four railroads, the Northern Pacific, Great Northern, Chicago, Milwaukee & St. Paul, and the Sault St. Marie, as shown in Fig. 1.

III.

CHARACTER OF THE TOWNS WHERE THE POWER IS TO BE MARKETED

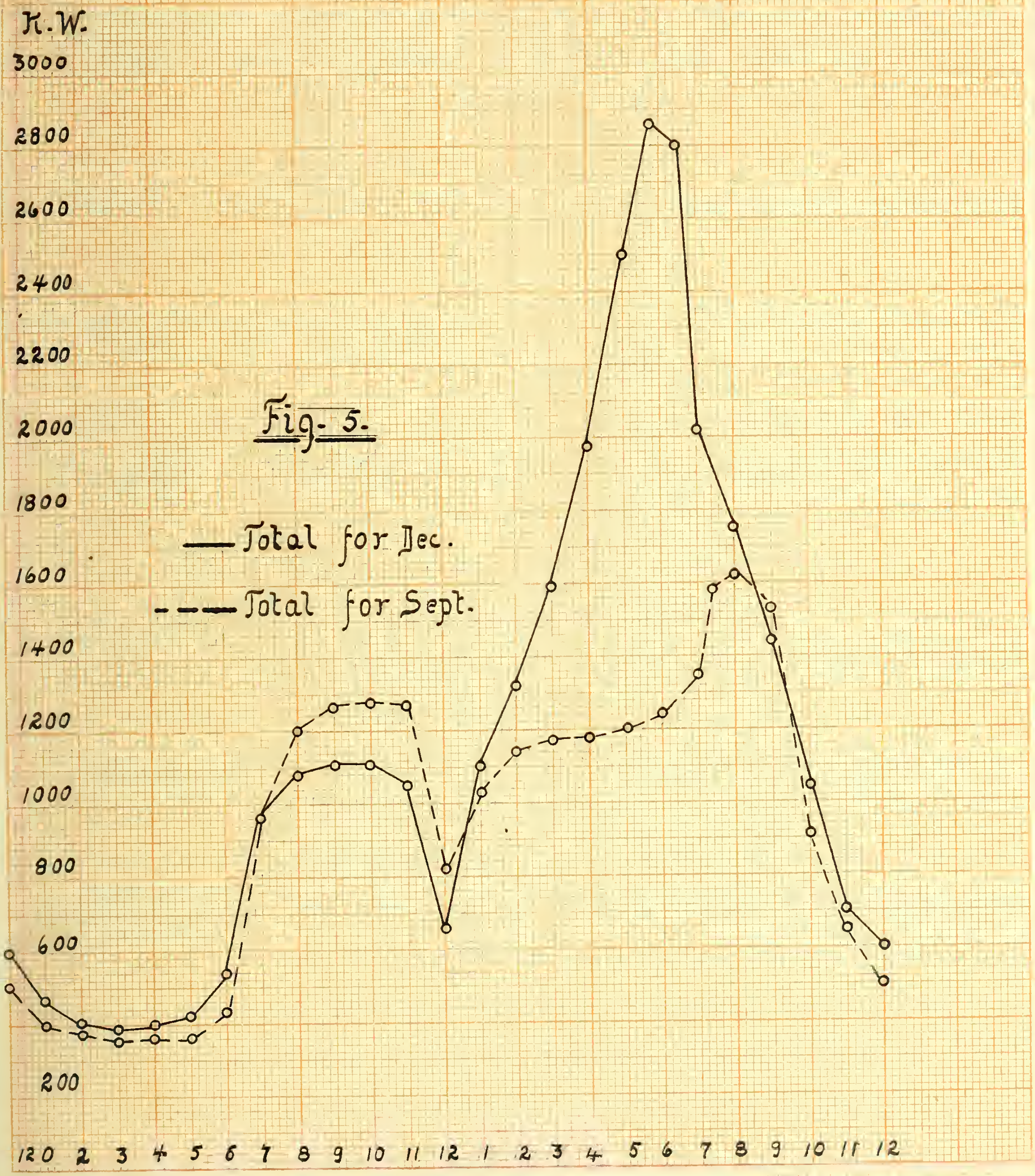
As stated above the distributing systems of Fergus Falls, Breckenridge and Wheaton are private owned, due to the fact that steam-power plants have been in operation in these places for a number of years and with the increasing demand for power these plants were not expanded because of the lower rates of the Ottertail Power Company and are now mostly controlled by them, the distributing systems being still partly in private hands. The steam-power plants are used only as reserves by the company.

At Fergus Falls the power for the industries of the city is largely received from private dams, as in the case of the mills and breweries, generating sufficient power for their own use only and because of the loss incurred by discontinuing the use of their own installed machinery which, if pt on the market would have little value, the possibility of supplying these with power, at least for a nunber of years to come, is very limited. The maximum lighting load will occur in December at about 5:30 p. m., and in September at 7:30 p. m.

Due to the fact that there are few industries and the population of the cities are mostly merchants and of a well-to-do

class, the lighting power consumed is very high in comparison to the population in all the towns.

At Breckenridge the machine shops of the G. W. R. R. are run with steam power, but numerous machine shops, elevators and mills receive their power from the company. The consumption of light in Breckenridge is comparatively not so great in the residence portions but is largely made up by the 14 saloons, breweries and ware houses. The street lighting in both Breckenridge and Wahpeton is quite extensive, consisting in 28-100 c. p. tungsten lamps for the principal street illumination and 26 arc lamps for the residence districts besides a number of arcs for lumber yards and ware houses, for Breckenridge, and 48-100 c. p. tungsten suspension lamps placed 100 ft. apart on either side of the street and around the courthouse and city hall, for Wahpeton. Here the mill, elevators, machine shops, garages and sawmills all receive their power from the company and including the smaller consumers, motors to the amount of 140 K. W. have been installed; assuming 60% of them to be in operation at the same time, we have 84 K. W. as the max. power consumption for the city. At the mill and machine shops the load amounting to 60 K. W. will be fairly constant from 8:00 a. m. until 6:00 p. m. dropping to a minimum at noon and to practically no load from 11:00 p. m. until 5:00 a. m. The max. power load will be in the spring when the sawmills are running and in the fall when the grain elevators are in operation from 8:00 a. m. until 6:00 p. m. Summing up the power load, we find that the power curve would be fairly constant until noon, increasing slightly toward evening and reaching its max. at about 5:30 p. m., drop-


K.W.
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
200
Fig. 5.
Total for Dec.
Total for Sept.
12 0 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12

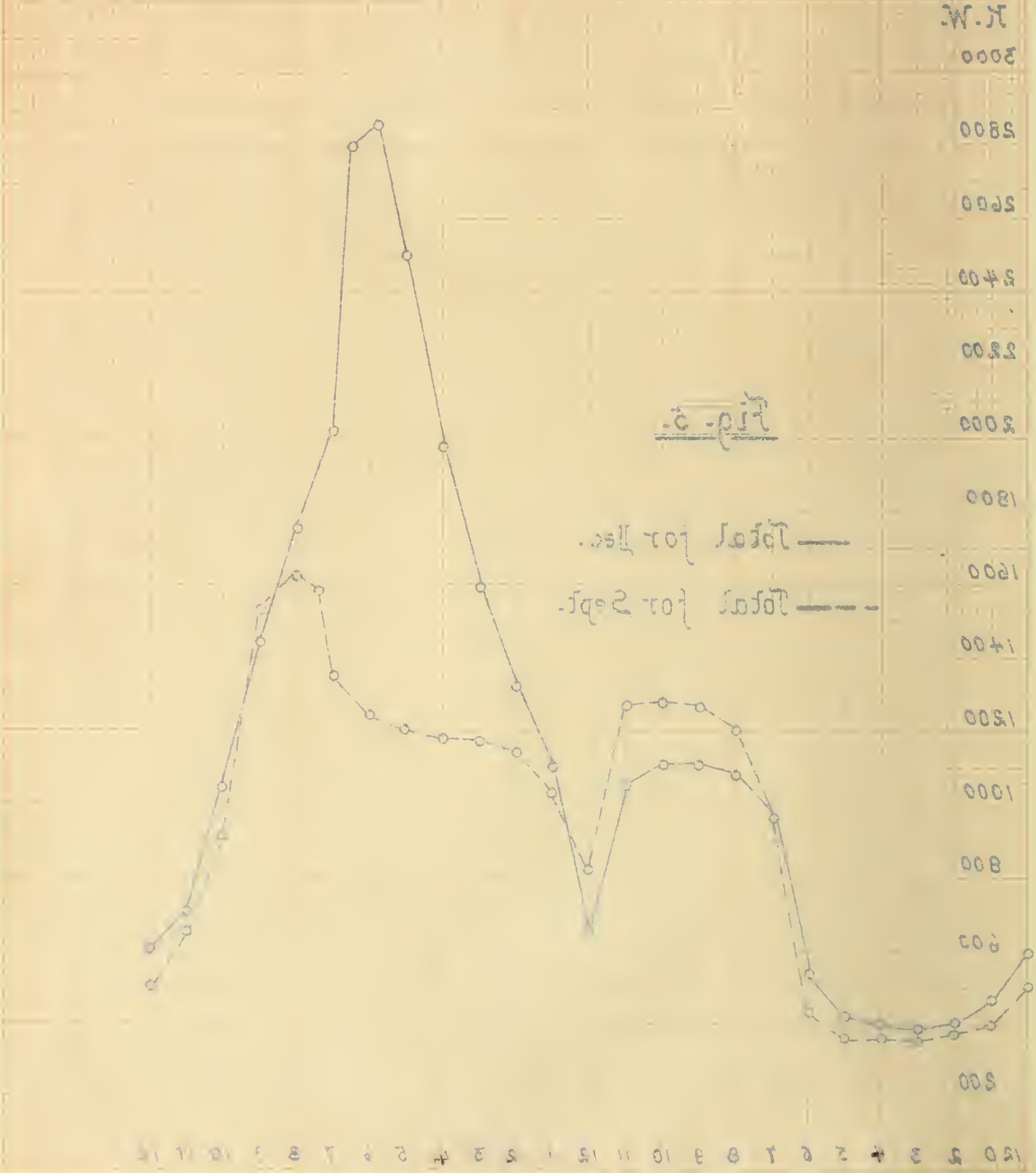

K.W.
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
200
Total for Dec.
Total for Sept.
12 1 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12

Fig. 5.

ping to a minimum during the night hours.

Campbell is the junction of the Soo and G. N. R. R. and its lighting load consists mainly in tungsten lamps for the illumination of the R. R. yards and street illumination. Power is supplied to two elevators and the R. R. repair shops. Knowing the load curves of the towns where power plants have been and computing curves for both power and lighting for each town from the conditions prevailing there and summing up the curves for each town and totaling these, we have the total load of Fig. 5. The curve shows a max. of 2900 K. W.

IV.

A. COMPUTATION OF THE AVAILABLE POWER FROM THE RUN-OFF

The statistics of the United States Geological Survey from which the following data was taken were collected at the station one mile below the entrance of the Pelican River. This data extends over a sufficient length of time to enable us to compare the different temperature, wind and rainfall conditions on which the flow of the Ottertail River depends and to estimate the possibility of power development.

The variations in the mean annual and monthly precipitation (in full lines) for the Red River Basin and also (in dotted lines) the mean annual and monthly precipitation for the Ottertail River Drainage Area for a number of years are given by the curves of Fig. 2. Curves for temperature variations, maximum, mean and minimum are shown in Fig. 6 covering a number of years. Comparing the curves of Fig. 2, for the precipitation of the entire Red River Basin of which the Ottertail Drainage Area is only a very

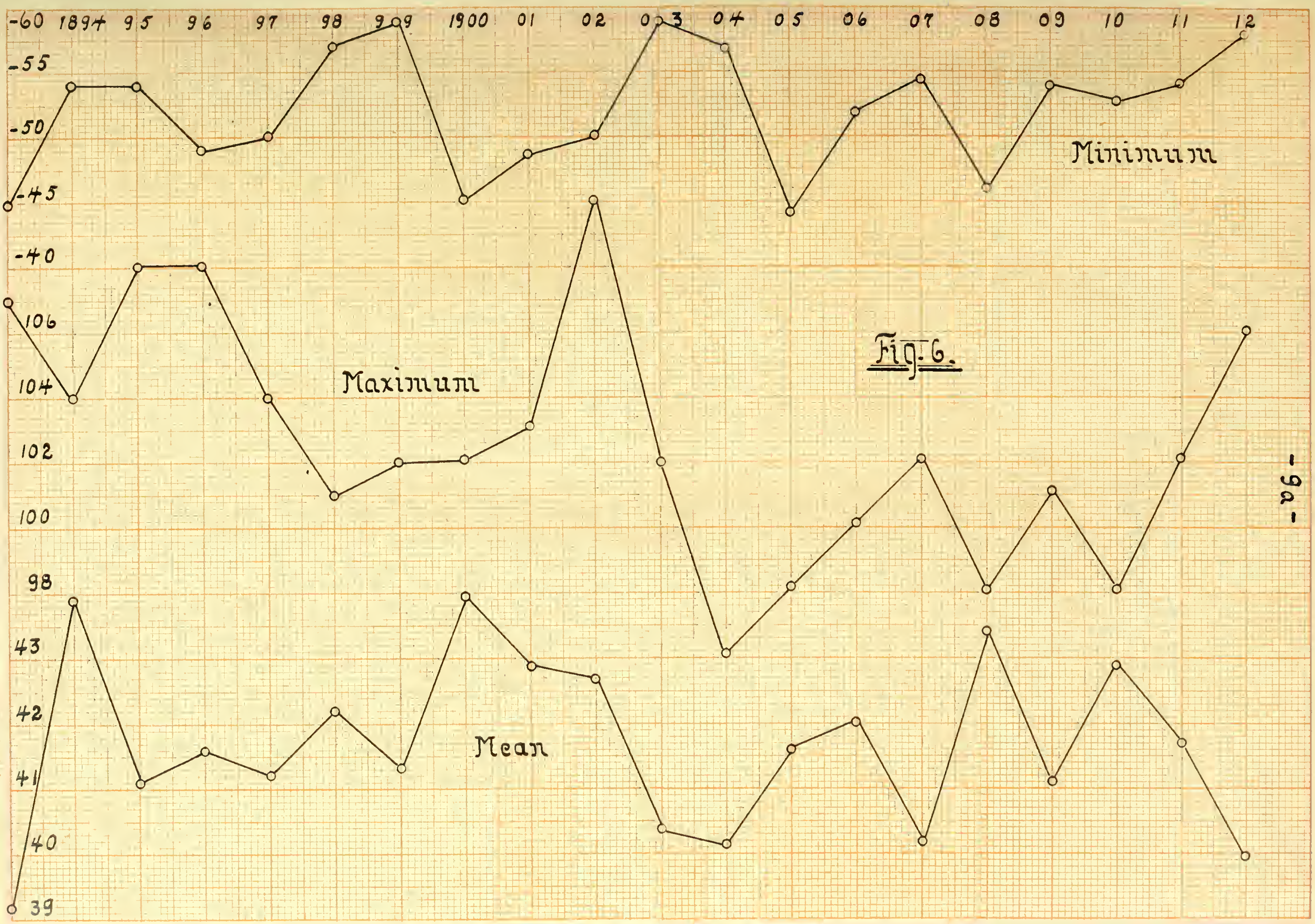

-60
1894
95
96
97
98
99
1900
01
02
03
04
05
06
07
08
09
10
11
12
-55
-50
-45
-40
106
104
102
100
98
43
42
41
40
39
Minimum
Maximum
Mean

Fig. 6.

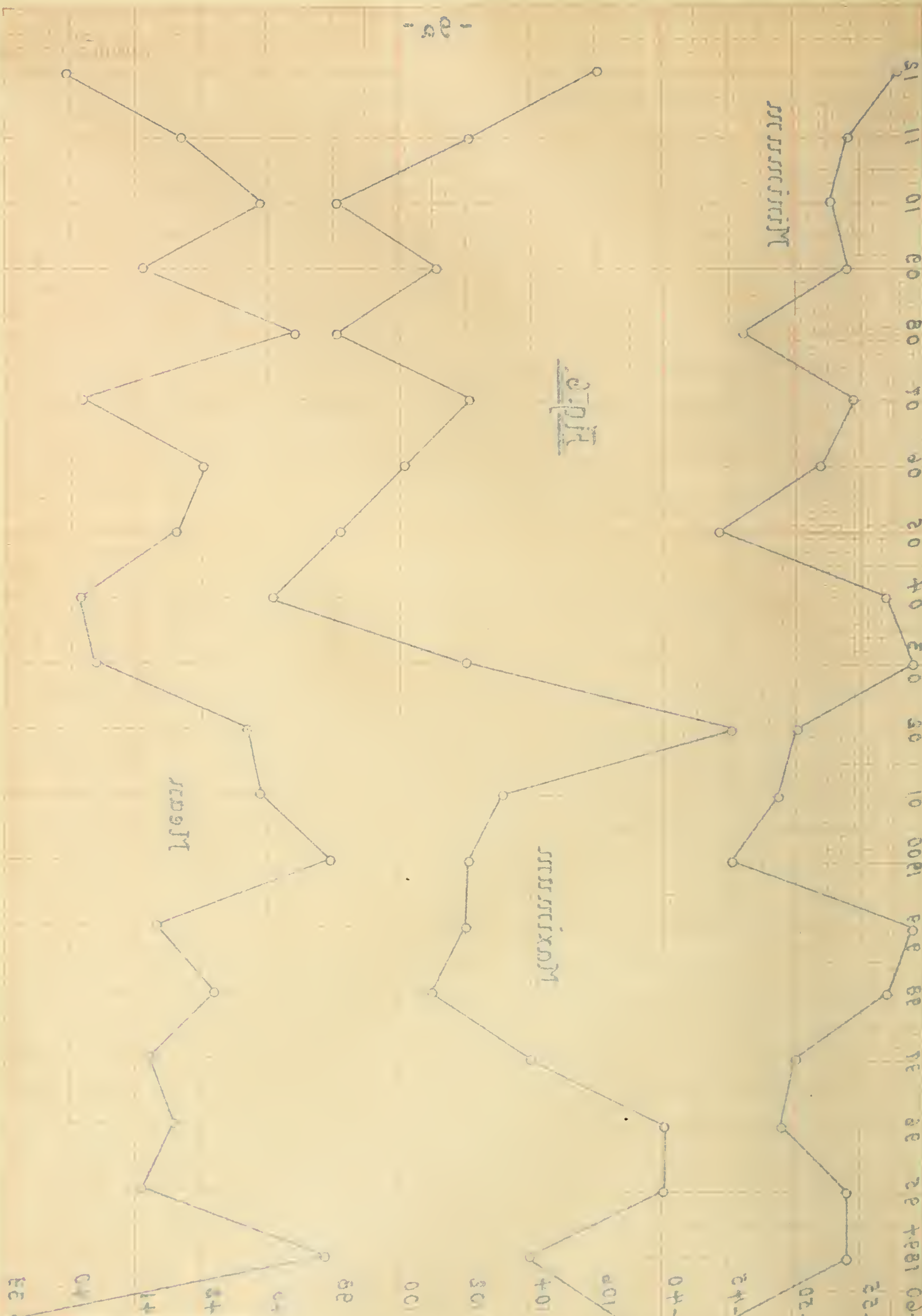


Fig. 6
Maximum
Minimum
Mean

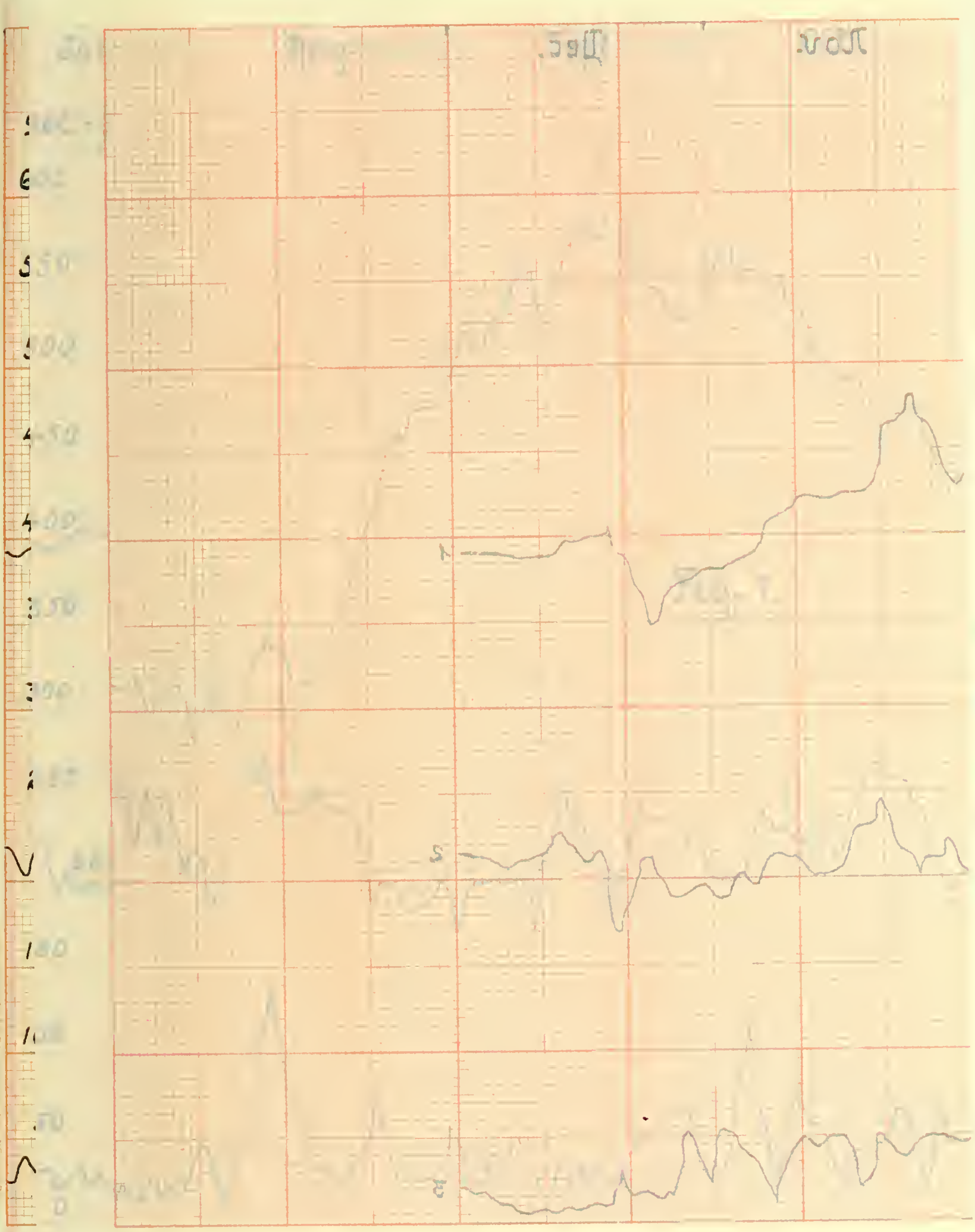
small part, with the precipitation curves of the Ottertail Drainage Area, we find that they run comparatively the same, altho the precipitation in the Ottertail Drainage Area is considerably larger, only once dropping below 23 in. per annum. The monthly precipitation curves also run the same, having the same variations during the same period of time. This tends to show that local winds, temperature and rainfall have little influence on the variation of the flow but do increase the volume.

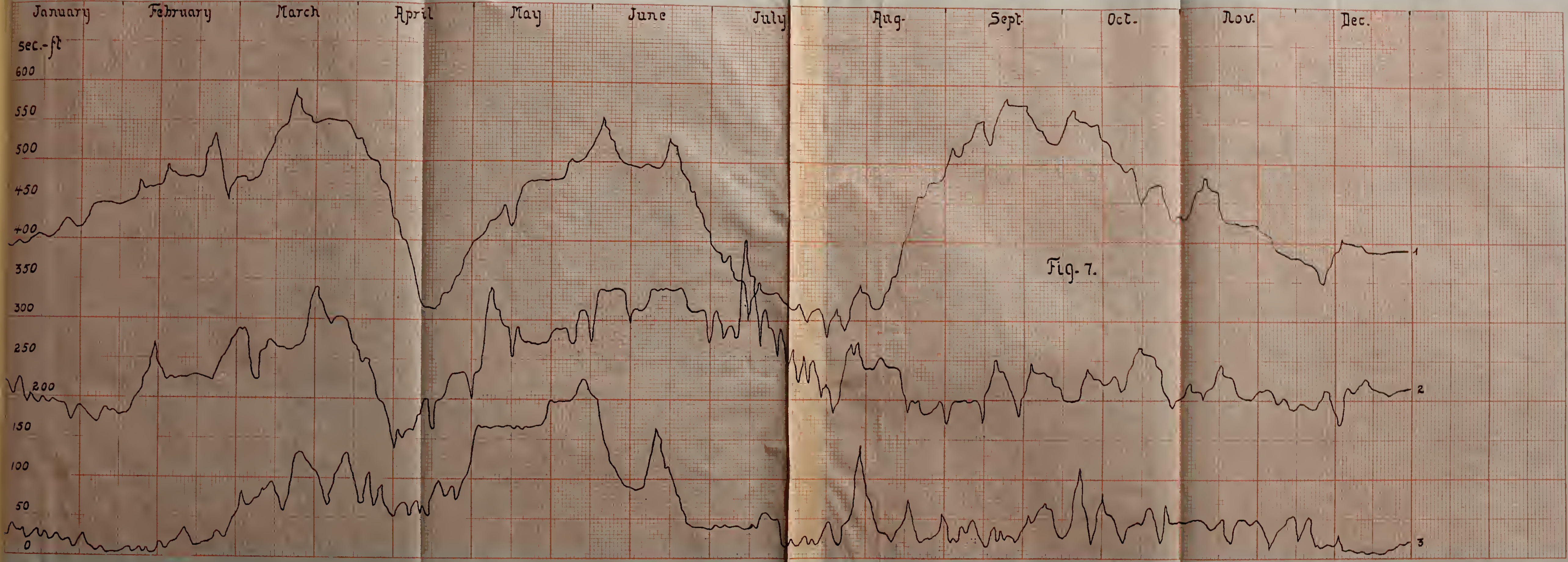
In Fig. 7 we have three hydrographs. 1/ for the year 1909 which was a fairly normal year, having a maximum temperature of 102°, a minimum of -45°, and a mean temperature of 41.6°.

2/ is a hydrograph of the Ottertail River for 1912, which is considerably below the normal, having an average precipitation of only 22.45 in., which, as can be seen by Fig. 2 has occurred only three times in a period of thirty years and only once in a period of twelve years in the Ottertail River Drainage Area. The maximum temperature was 106°. minimum temperature -53° and mean 39.9°. The year was exceptionally windy, the maximum wind velocity being 80 miles per hour.

3/ is a hydrograph of the Pelican River for 1912 which shows that the variations in flow of this river and the Ottertail River are similar, both reaching their maximum and minimum at about the same time.

If the average run-off for May of the entire drainage area is 284 sec. ft. and the mean run-off per square mile for the same month is 0.127 sec. ft., the scale for the run-off in sec. ft. per sq. mi. will have a value of 0.217 where the scale for the




January
February
March
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
sec.-ft
600
550
500
450
400
350
300
250
200
150
100
50
0
1
2
3
Fig. 7.

entire drainage basin run-off has a value of 284. In the same way, if the average run-off of the entire drainage basin for June is 325 sec. ft., and the mean run-off per sq. mi. for June is 0.248 sec. ft., the scale for the run-off in sec. ft. per sq. mi. will have a value of 0.248 where the scale for the entire run-off for June has a value of 325. The difference on the curve paper is 8 mm. or:

$$8 = (0.24 - 0.217) = 1:X$$

$$X = \frac{0.031}{8} = 0.004$$

or 1 mm. is equal to 0.004 sec. ft. per sq. mi. By reading off the values of the hydrograph for this scale we find for 1 sec. ft. discharge per sq. mi.:

$$\frac{1810.56}{8.8} = 11500 \text{ theoretical h. p.}$$

can be developed, or as the run-off for December was 0.16 cu. ft. the minimum run off during the entire year:

$$0.156.11500 = 1800 \text{ theoretical h. p.}$$

assuming a 56 ft. head. Taking the hydraulic efficiency to be 0.80 the actual h. p. would be:

$$1800.0.80 = 1450 \text{ h. p.}$$

B. SELECTION OF UNITS

As the year 1912 gives a minimum flow and almost the poorest conditions to be expected this year will be selected for the computation of the units to be installed and for the arrangement of the entire plant.

From the survey map Fig. 8 of the Ottertail River we find that a dam 56 ft. in height would bring the water level to

Fig. 8.

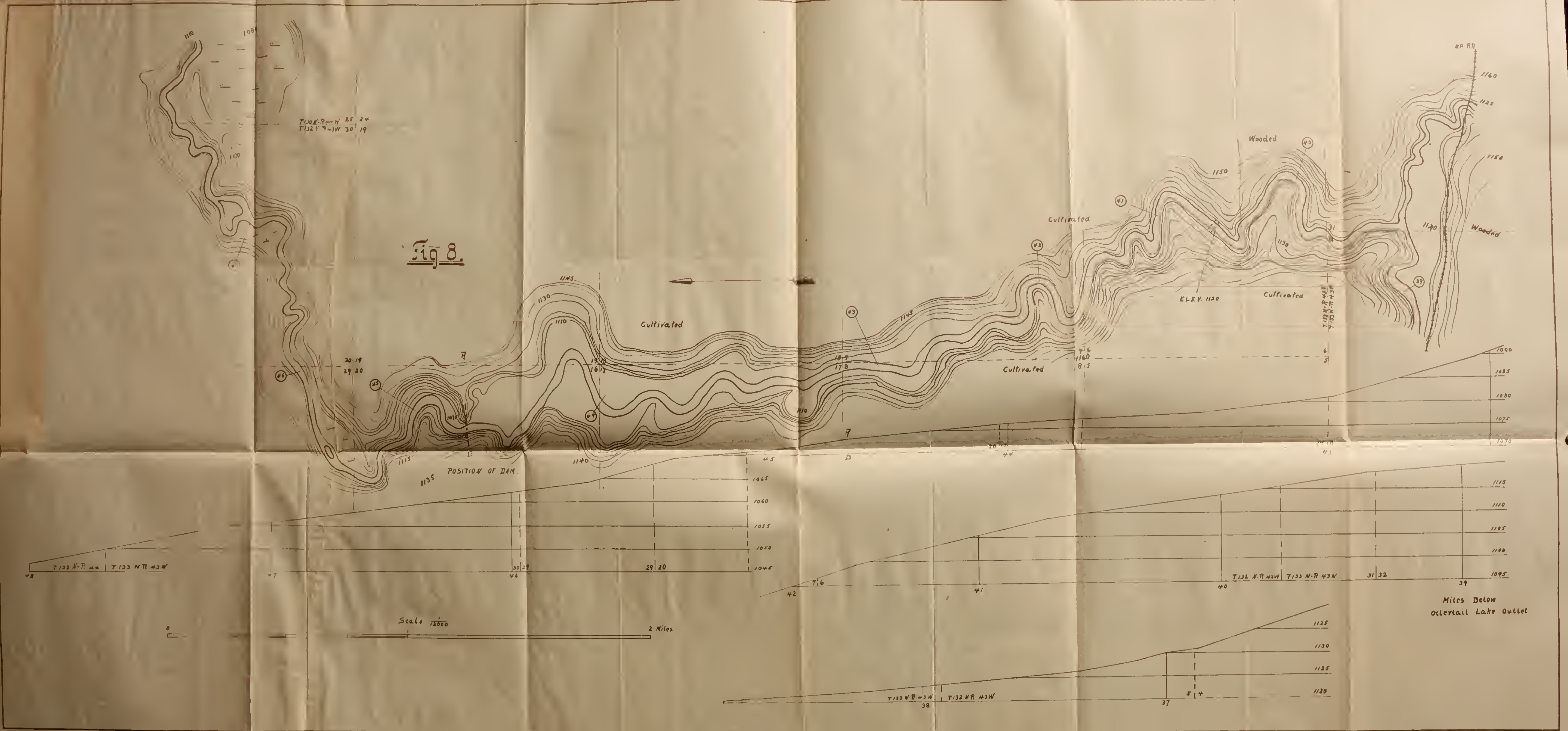


Fig 8.
Wooded
Cultivated
Cultivated
Cultivated
Cultivated
Wooded
ELEV. 1120
POSITION OF DAM
Scale 1/12000
2 Miles
T132 N-R 43W
T133 N-R 43W
Miles Below
Ottertail Lake Outlet

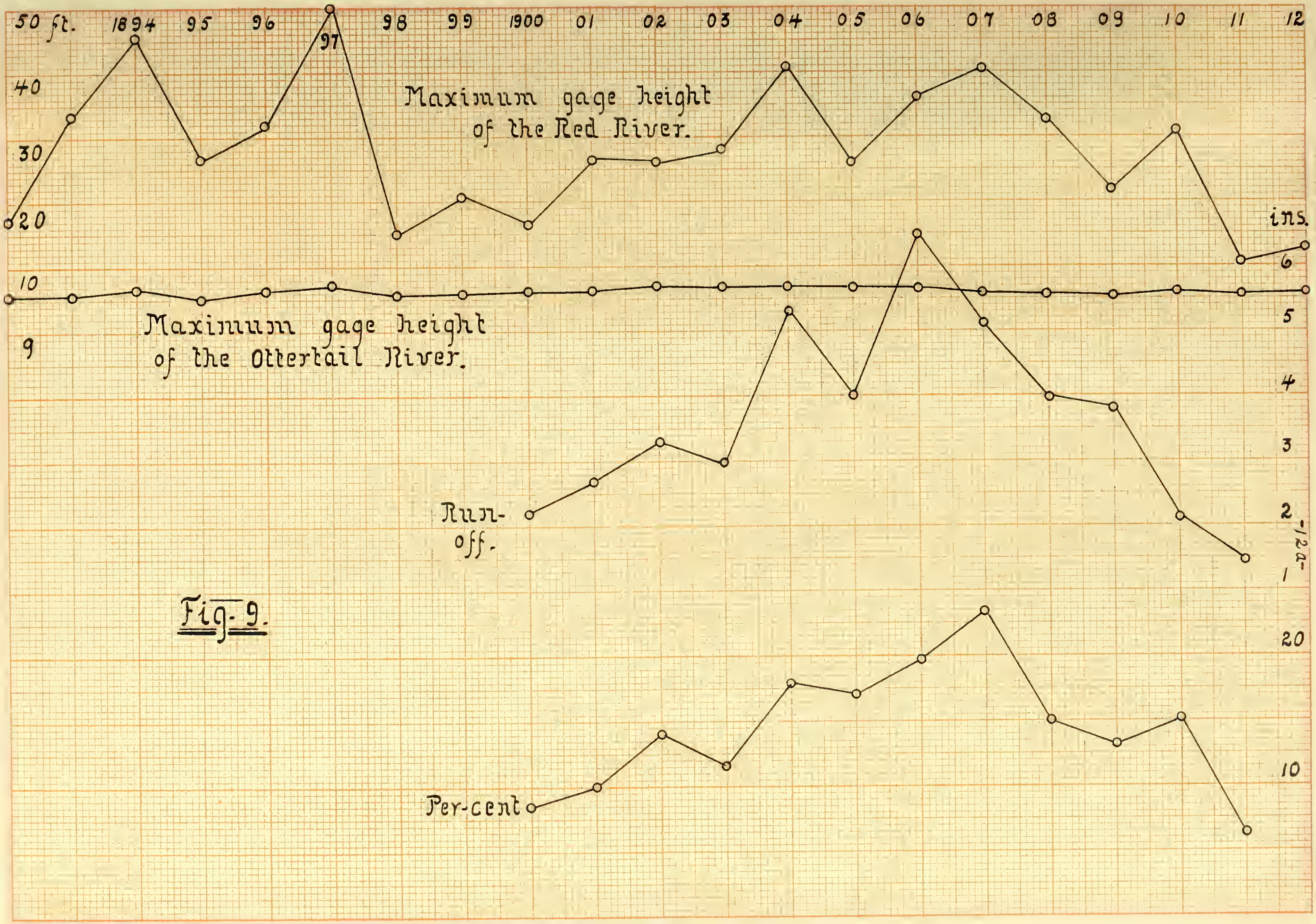
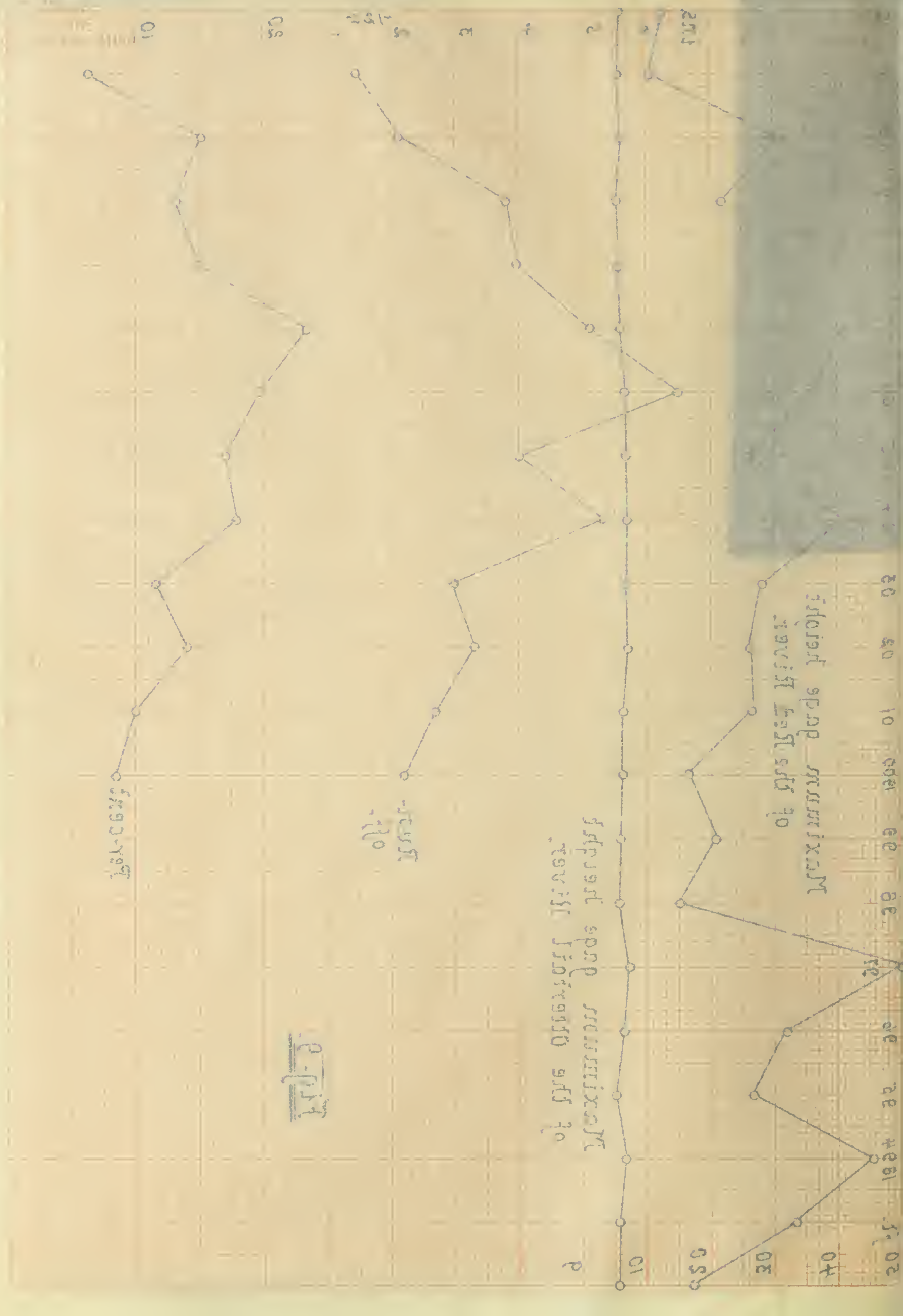

50 ft.
40
30
20
10
9
1894
95
96
97
98
99
1900
01
02
03
04
05
06
07
08
09
10
11
12
ins.
6
5
4
3
2
1
20
10
Maximum gage height of the Red River.
Maximum gage height of the Ottertail River.
Run-off.
Per-cent

Fig. 9.



elevation, 1128 leaving 5 ft. for the maximum rise of water in the reservoir which, as can be seen from the curves of gage height and variations Fig. 9, will be ample. The small variation in gage height of the Ottertail River is due to the fact that the upper part flows thru many lakes, which act as reservoirs for the surrounding country and in case of showers gives the water a chance to spread out and flow away gradually. In the last twenty years the maximum variation of gage height has not exceeded 2 1/2 ft. as can be seen by the curves. The lower part of Red River, however, passing thru rolling prairies and clay soil, is subject to much greater variations due to spasmodic showers. A curve of gage variations for the Red River is also given in Fig. 9. Bringing the crest of the dam to elevation 1128 would necessitate the raising of the highway bridge below the entrance of the Pelican River and the construction of an approach to elevation 1140. The water of the storage area would back up as far as T133N - R43W and would approximately cover 300 acres of land, all of which is the river channel, covering no cultivated lands nor wooded territories and would endanger no private property or land during periods of high water. Corewalls and earthwork embankments would have to extend to the contour lines 1135 on each side. These, however, would be the only retaining walls necessary for the storage of the water.

From the total load curve we see the maximum to be 2850 K. W. At the Hoot Lake plant 1800 K. W. are generated and at Fergus Falls 650 K. W. The Hoot Lake plant consists in .3-600 K.W. generators and as this plant and the proposed one are to work

together, and the daily load curve corresponds to this arrangement, it seems advisable to install three units of equal capacity, one being large enough to make up for the present overload and eventual increase in load for a few years to come, arranging the power house so that the remaining two units may be added as soon as the demands for power has increased to that extent. To ascertain whether this arrangement is possible, the storage capacity must be investigated and compared with the load conditions.

As our minimum flow is 200 sec. ft. = $5.66 m^3$ per sec. and as we are able to store water for about 8 hrs. a day by allowing the Hoot Lake plant to carry the night load, we can have 1.88 m^3 per sec. more at our disposal, or a total of $7.54 m^3$, including the water stored when only one turbine is in operation. The following gives an approximation of the water taken by the turbines if the above conditions are to be fulfilled:

No. of units in Operation	Time	m^3 (each) per sec.	Total Consumption
1	7 a.m. - 2 p.m.	4	100,000 m^3
2	2 p.m. - 4 p.m.	4	57,600 "
3	4 p.m. - 7 p.m.	77	130,000 "
2	7 p.m. - 11 p.m.	77	115,000 "
Total	24 hours		402,600 "

The storage for 24 hours would be: 480,000 m^3. Using an exciter turbine taking about 32,500 m^3 we would have 44,900 m^3 for the fishway and as margin for excess load.

Taking then $4m^3$ for each turbine unit we would have

the theoretical h. p.:

$$Na = 1000\ Q.H.$$

$$= \frac{1000.4.17}{75} = 900 \text{ h. p.}$$

where Q is the quantity of water and H the available head in meters. If we let the loss sustained thru friction by the water passing thru the flume and turbine be ρ H. the available head would be $H-\rho H = H(1-\rho) = \varphi H$. In the same way let the loss thru the water leaving the turbine with a certain small velocity be δH and the frictional losses in the bearings, etc., be μH. The total loss of head would therefore be:

$$H\ (1 - \rho - \delta - \mu)$$

and the effective capacity of the turbine:

$$Ne = \frac{1000\ Q.\ H.\ (1 - \rho - \delta - \mu)}{75}$$

Assuming $\varphi = 0.13\%$; $\delta = 4\%$ and $\mu = 1\%$, the efficiency would be $n = 0.82\%$

$$Ne = 900.0.82 = 750 \text{ h. p.}$$

The capacity of all three units is then 2250 h. p. and would give us three 500 K. W. generators with some margin of capacity for overload.

V.

SELECTION OF A DAM

The costs of a gravity type dam and a reinforced concrete dam are to be compared and the feasability of using the one or the other is to be discussed. Both dams must needs have a 60' spillway and earth dikes extending 1320 ft. on one side and 320 ft. on the other side. Fig. 10 gives a sectional view of the

gravity type dam.

A. DAM OF GRAVITY TYPE

The rubble concrete dam has a spillway of 60 ft. flanked on either side by abutments of the same material and earth dikes 1320 ft. and 320 ft. respectively. The dam itself would have a height of 56 ft., the abutments 61 ft. and the corewalls (dikes) with side slopes of 1 to 12, and 1 on 4 with a 4 ft. berm and an average height of 14 ft., for the earth embankments. The actual work of diverting the river would be accomplished by building a rock filled cofferdam above the site selected for the permanent dam with a maximum height of 16 ft., the natural bank forming one side, thereby diverting the river into one half of the channel and allowing the excavation to be carried on in the dry bedrock. A concrete mixing plant would be placed on the bank and a crushing plant at a favorable place near by where a quarry would be opened. All sand and gravel would be taken from nearby pits and carried to the chutes of the mixing plant by dump cars, the cement would be carried by barrows. The mixtures would be in 1 cu. yd. batches in the preparation 1:2 1/2:5 using Portland cement. The output would be about 130 cu. yds. per day and the delivery consist of 1 cu. yd. tipping buckets and placed in the forms by means of push cars and a 40 ft. boom derrick operated from a trestled platform by drum engines. The successive fills would be bound by the projection of large stones, not exceeding 0.75 cu. yds., half and half into each fill, care being taken to bring the fills into horizontal layers over the entire surface of each section. The dam would be constructed in two sections, bound together with vertical

keys 2 1/2 ft. apart and terminating 2 ft. from the top. After the first section is completed the cofferdam would be removed and built across the other channel sending the water thru three 10 x 10 ft. sluiceways left in the structure. The mixing plant would be removed to the other side of the river and the same forms would be of 2 in. dressed pine planks braced with 4 x 6 in. studding spaced 3 ft. apart and stiffened by 6 x 8 in. horizontal beams. The forms would not be interchangeable, but would be knocked down as the first section was stripped. and rebuilt for the second section. The earth dikes would be filled by drag and wheel scrapers drawn by mules, maintaining an equal fill on either side to keep unnecessary strain from the corewall. Clay puddle and riprap would keep the walls from erosion.

The camp would consist of the following buildings:

2 dormitories for 60 men	3500 sq. ft.
2 mess halls for 60 men	3000 " "
3 individual houses for 3 men	600 " "
1 storehouse	1000 " "
1 machine shop	400 " "
1 blacksmith shop	100 " "
Total	8600 " "

Cost of construction:

6500 ft. B.M. of lumber at $18.50	$12.00
12 carpenters for 30 days at $3.00	10.80
17000 sq. ft. tar paper at 0.0225	3.80
Nails, etc.	.75
Total	$27.35

Cost per sq. ft. of building 0.318.

The excavation and screening of gravel would require:

1 screening plant

5 wheel scrapers

6 spans of mules and harnesses

5 living tents

2 horse tents

dinkey engine, 1/4 mile track, 5 dump cars.

The investment cost would be about $8000 and the daily plant charges:

Interest and depreciation at 35%	$28 00	$46.80
Coal for boiler		2.00
Coal for engine		.50
Oil for engine		.50
Feed and care of mules		6.00
	Total	$55.80

The plant for quarrying and crushing the rock would be:

1 crusher

1 hoisting engine and boiler

1 derrick (40 ft.)

2 steam drills

3 dump cars

1 blacksmith shop

1 winch

The investment would be about $4000 and the daily plant charges:

Interest and depreciation at 25%	$16.00	$26.70
Coal for boilers		3.00
Oil for engines		.20
Explosives		15.00
	Total	$44.70

The mixing plant consists of 3 mixers, 3 cement trucks, 100 ft. of track with trestle, 1 sand chute, 2 sand cars, 1 cement house. The cost of the plant would be about $3000 and the daily plant charges:

Interest and depreciation at 20%		$10.00
Coal		1.35
Oil		.07
130 cu. yds. at 0.088	Total	$11.42

The plant for placing the concrete would be:

3 hoisting engines and boilers, 4 derricks, 6 tipping buckets, 3 cars, 2 dump cars, 15 wheel barrows and 20 shovels.

The investment cost would be $4000 and the daily plant charges:

Interest and depreciation at 35%,	$14 00	$23.30
Coal		3.00
Oil		.50
Total 130 cu. yds. at $0.11		$26.80

Labor rates would be about as follows:

Foremen	$3.00 to $5.00
Engineers	2.25 to 3.50
Firemen	1.75 to 2.75
Tagmen	2.60 to 3.50
Carpenters	2.00 to 3.50

Rivermen	$3.00
Electricians	3.00
Riggers	2.50 to $3.50
Mechanics	2.75 to 3.50
Cooks	2.00
Laborers	1.75 to 2.25
Water boys	1.50

MAIN DAM AND CONCRETE

The cost of the 11700 cu. yds. of rubble concrete in the main dam and corewalls in place inclusive of labor and plant charges would be:

	Foremen	Labor (skilled)	Labor	Cost cu. yd.
Stone	1	4	4	$1.25
Sand	1	2	5	.45
Forming	2	12	1	.65
Mixing	2	2	20	.55
Placing	1	3	15	.70
Cement				2.30
Total	7	23	45	$5.90

The lumber for forms if used twice for the dam and then for the corewalls would average about 18¢ per sq. ft. of surface.

CONCRETE COREWALLS, NORTH DIKE

This corewall would average about 14 ft. in height, being 2 ft. thick at the top and having a batter of 1 in 12 on each side, founded in bed rock. The concrete would be delivered by the dinkey engine on rail and trestle where needed, to a distance of

1320 ft. The cost of the corewall would be as follows:

3000 lbs. of cement at $2.30	$6900
700 cu. yds.of sand and gravel at .75	525
1400 cu. yds. broken stone at $1.25	1700
Mixing concrete (2100 cu. yds. at .55)	1150
Placing concrete (21 cu. yds. at .80)	960
Total	$13335

Cost per cu. yd. including excavation $6.48.

SOUTH DIKE

The length of this corewall would be 320 ft. with the same batter as the north corewall and a volume of 500 cu. yds. Due to the smaller distance to which the concrete must be carried the cost per cu. yd. can be assumed to be $5.75. The entire cost of the corewalls would be $16475.

EARTHWORK

The volume for both dikes would be approximately 54000 cu. yds. at 40 cts. per cu. yd. and would cost approximately $21600.

POWER HOUSE

The power house in place including labor, excavation, material, depreciation of machinery and interest on investment would cost about $10000.

B. THE REINFORCED CONCRETE DAM

The dam would be erected in practically the same way as the rubble concrete dam and in comparing the two the same daily output of the mixers and equipment will be assumed with

small variations in labor and material. In determining the dimensions and material of the deckspans and supporting arches the whole is to be regarded as sectional concrete slabs with uniform distribution of pressure resting on reinforced concrete beams and the dimensions for such determined at intervals of 16 ft. The dimensions of the concrete spans, beams and steel reinforcing will then be selected for these intervals.

Nomenclature

m^3 = cubic meters.

Na = absolute or theoretical h. p.

Ne = effective h. p.

e = specific ductility.

l = length

λ = length of extension

α = coefficient of ductility

$\sigma = \frac{P}{F}$

p = force or load

F = area

E = moduli of elasticity

Ee = moduli of elasticity for steel

E_b = moduli of elasticity for concrete

σ_e = tensile strength of steel

σ_c = " " " concrete

M_c = bending moment

q = load per unit length

Q = total load

$R = \frac{Q}{q}$

kg/m^2 = kilograms per square meter

kg/m^2 = " " " cm.

V = vertical stress

D = horizontal pressure

z = " tensile stress

C = width

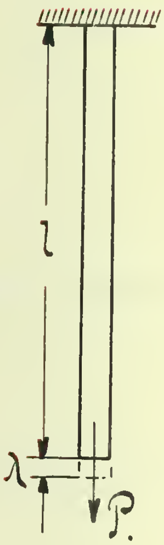
h = heighth

γ = coefficient dependent upon the ratio $\frac{C}{h}$

at. = atmosphere = 760 mm. pressure

a. FUNDAMENTAL EQUATIONS IN REINFORCED CONCRETE

The elongation of the beam in the accompanying figure would be:



$$E = \frac{\lambda}{l} = \alpha \cdot \sigma$$

where $\sigma = \frac{P}{F}$ or as $E = \frac{1}{\alpha}$

$$E = \frac{\lambda}{l} = \frac{\sigma}{E} = \frac{P}{E.F.}$$

Experience has taught that most satisfactory results are to be had with E_b = 140,000 at. and E_e = 2100000 at. or

$$n = \frac{E_e}{E_b} = \frac{2100000}{14000} = 15$$

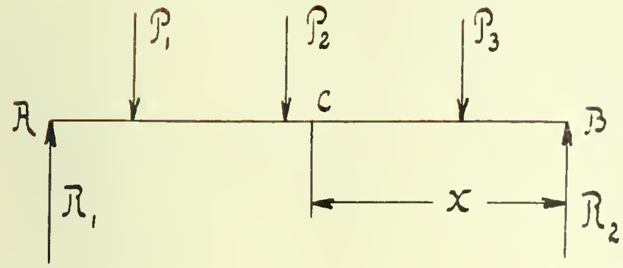
$\sigma_e = \frac{\lambda}{l} E_e$ for the steel, and

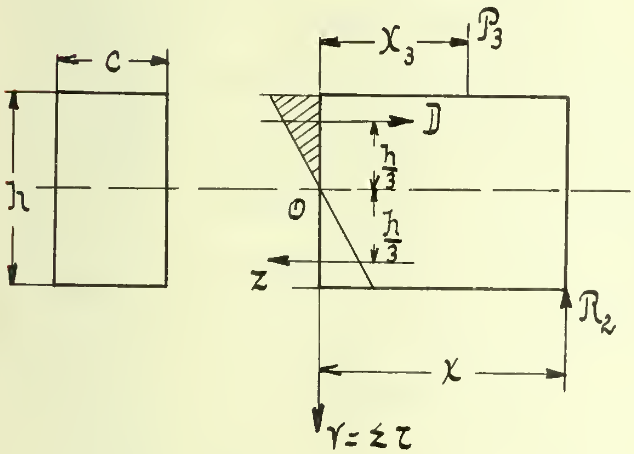
$\sigma_c = \frac{\lambda}{l} E_b$ for the concrete,

from which $= \frac{E_e}{E_b} = n$ or:

$= n. \sigma_c = 1000$ at.

b. RESISTING MOMENT





The forces R_1, P_1, P_2, P_3 and R_2 are working on the beam in the accompanying figure. If we cut the beam at a given point C we can bring this piece CB into equilibrium by assuming the internal stresses in this section to be external forces, D as the compressive force and z the tensile, we then have:

$$D - z = 0 \text{ or } D = z,$$

and the shaded areas would equal these forces:

$$D = z = 1/2\sigma_1 \frac{h}{2}.c = \sigma_1 \frac{b.h}{4}$$

The algebraic sum of all vertical forces would = 0:

$$V + P_3 - R_2 = 0$$

$$V = \Sigma(t) = R_2 - P_3;$$

likewise the static moment of all internal forces:

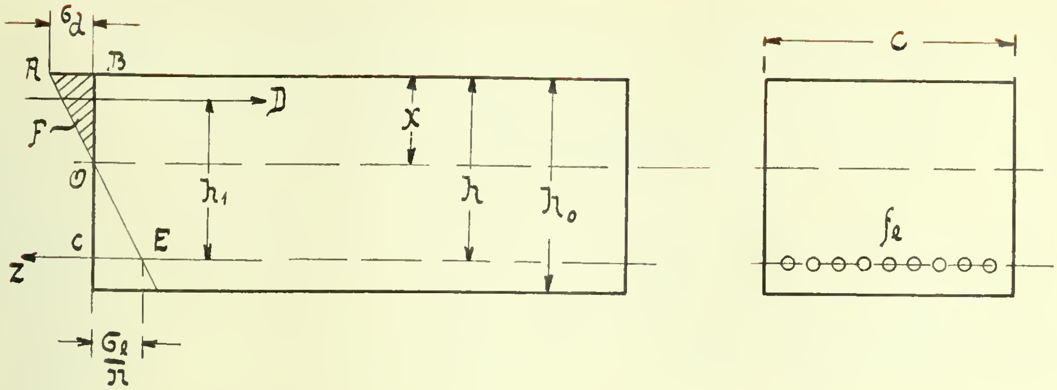
$$M_C = D\frac{h}{3} - z\frac{h}{3} = 0.$$

As D = z, we now have:

$$M_C = 2\,D\frac{h}{3} = 2\,\sigma_1 \frac{hh^2}{3} = \frac{bh^2}{6}\sigma_1$$

Let q equal the uniformly distributed load per unit length, then the greatest bending moment, at the center of a beam, is:

$$M_C = \frac{q.1^2}{8}$$



In the accompanying figure let:

σ_d = the greatest compressive unit stress of the concrete.

σ_e = the greatest tensile unit stress in the steel, -kg. sq. cm.

X = distance from top of beam to neutral surface in cm.

h = distance from center of steel to top of beam in cm.

h = effective depth in cm.

b = width of the beam in cm.

fe = area of all the steel reinforcing bars.

As the tensile stress of the steel is nø times greater than that of the concrete at the same point, CE will be $\frac{e}{n}$. The neutral surface will not be in the center of the cross section but is dependent on the ratio of σ_d to σ_e.

From the figure we see that:

$$\frac{x}{h - x} = \frac{\sigma_d}{\sigma_e : n} = \frac{n\,\sigma_d}{\sigma_e}$$

$$\frac{x}{x + h - x} = \frac{\sigma_d}{\sigma_d + \frac{\sigma_e}{n}}$$

$$x = \frac{\sigma_d}{\sigma_d + \frac{\sigma_e}{n}} \cdot h$$

As for example if $\sigma_d = 40$ at. and $\sigma_e = 1000$ at. $x = 0.375h$ or:

$$x = \alpha . h$$

$$D = \sigma_d \frac{C . x}{2}$$

$$h_1 = h - \frac{x}{3} = B.h$$

$$M_C = D.h = \frac{C.x}{2} . \sigma_d.h \quad \text{or if } \frac{M_C}{\sigma_d} = W$$

the resisting moment of the steel:

$$W = \frac{C.x}{2}. h_1 = \alpha. 1/2\ h_1^2 .C = \gamma.h_1^2 .C$$

From this we have also: $\gamma \cdot c \cdot h_1^2 = \frac{M_c}{\sigma d}$

$$h_1 = \gamma \sqrt{\frac{M_C}{C}}$$

As $z = D$ or $z = fe\sigma_e$ and $D = \frac{C.x}{2} .\sigma_d$

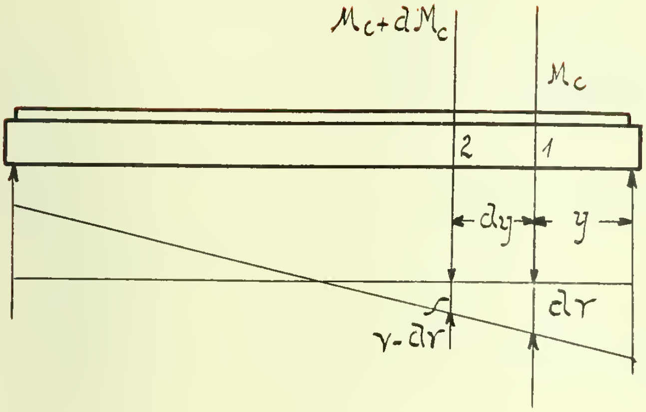
$$fe.\sigma_e = \frac{Cx}{2} . \sigma_d$$

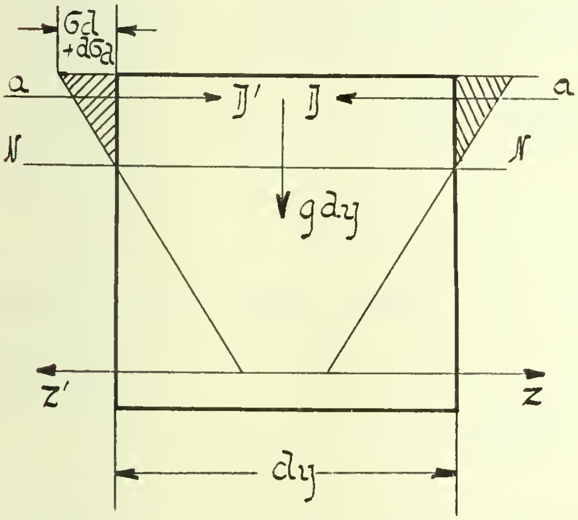
$$fe = \frac{Cx}{2} \quad \frac{\sigma_d}{\sigma_e}$$

$$fe = \varepsilon . b . h_1 \qquad \varepsilon = \varepsilon$$

The use of concrete, however, is only necessary as far as it is used to cover and surround the steel reinforcing. We therefore can assume the section AB as shown in Fig. 11, with the span of the beams at 15 ft.

The shearing unit stress v can be calculated with the aid of the accompanying figure, assuming a uniform distribution of load.





Two sections 1 and 2, dy from each other are acted on by a moment M_C and a shearing force v and the other section, by a moment $M_C + dM_C$, and a shearing force v - dv, regarding that part which lies above the line a a we can bring the stresses in section 1 equal to a compressive force D and those in section 2 equal to D^1. $D^1 > D$; $D^1 = D + dD$; $D^1 - D = D + dD - D = dD$, a force toward the right which produces a horizontal shearing unit stress in the beam. A shearing force in a-a will oppose this which, if τ_a = to the shearing unit stress and C the width of the beam, is equal to $\tau . C . dy$. Therefore:

$$\tau_a \cdot C \cdot dy = dD$$

τa will be a max. where dD (or D) is a max., or for that part for which the shaded area is the greatest. For N-N, D is equal to the entire area of the triangle and the greatest horizontal shearing unit stress must therefore occur at this point, therefore:

$$D = \frac{C.x}{2} \sigma_d$$

$$dD = \frac{C.x}{2} \cdot d\,\sigma_d \quad \text{and}$$

$$\tau . C . dy = \frac{C.x}{2} . d\sigma_d \text{ or}$$

$$\sigma_d = \frac{2 M_C}{C.x (h - \frac{x}{3})} ; d\sigma_d = \frac{2}{Cx (C - \frac{x}{3})} dM_C$$

and
$$\tau . C . dy = \frac{Cx}{2} . \frac{2}{Cx (h - \frac{x}{3})} . dM_C$$

$$\tau = \frac{1}{C(h - \frac{x}{3})} \quad \frac{.dM_C}{dy}$$

The equation of moments for point O is:

$$M_C + dM_C + q.dy.\frac{dy}{2} - M_C - v.dy = 0$$

$$V.dy = d.M_C + q \frac{dy^2}{2}$$

$q.\frac{dy^2}{2}$ is infinitely small in comparison to the rest and can be dropped.

$$V = \frac{dM_C}{dy}$$

(This is also proof that the greatest stress is in that section where the shearing force is = 0.)

$$\tau = \frac{1}{(h - \frac{x}{3})C} \cdot V \text{ and as } h_1 = h - \frac{x}{3}$$

$$\tau = \frac{V}{h_1 . C}$$

The greatest shearing force would be:

$$\tau \max = \frac{R}{h_1 . C}$$

The bond stresses are z and $z^1 = z + dz$, and their difference $z^1 - z$ is a force toward the left. The strength of the bond is equal to the area times the bond stress. Let U = the area of all the steel and dyU the area of the steel for the small beam

piece dy and τe the bond stress, then:

$$\tau e \,.\, U \,.\, dy = z' - z = dz$$

now $z = fe.\,\sigma e$, $dz = d.\ fe\,.\,\sigma e$, and therefore

$$\tau e \,.\, M \,.\, dy = fe \,.\, d\sigma e$$

σe was equal to $\dfrac{Mc}{fe\,(h - \frac{x}{3})}$ and $d\sigma e = \dfrac{d - M_C}{fe\,(h - \frac{x}{3})}$

therefore $\tau e.U\,.\,dy = fe\ \dfrac{d\ M_C}{fe\ (h - \frac{x}{3})}$

$$\tau e = \frac{1}{h_1\,.\,U} - \frac{dM_C}{dy}$$

$$\tau e = \frac{V}{h_1\,.\,U}$$

For V max. = R

$$\tau e\ \max. = \frac{R}{h_1\,.\,U}$$

As concrete can only stand a shear of 4.5 at. at the most, the excessive strain is taken up either by bending the ends of the reinforcing rods under an angle of 45° or by inserting vertical stirrups. If

$T = \dfrac{(\tau\max. = 4.5)^2}{\tau\max.}\ \dfrac{C_1\,.\,1}{4}$ is the excess strain,

the area of the steel rods to be bent is:

$$fe' = \frac{z}{\sigma e} = \frac{F}{V2.\sigma_e} = 0.707\ \frac{T}{\sigma_e}\ \ cm^2$$

and the number i of the rods to be bent determined.

The number of stirrups can be determined:

$$ft = \frac{T}{\tau e} \quad \text{where } \tau_e = 4/5\,\sigma_e$$

In practice both ways are used, 50% of T being taken by the stirrups and 80% by the bent rods.

The excess pull which is not taken by the bond between

steel concrete is taken up by the ends of the rods being bent at right angles. We have:

$$P = \frac{(\tau max - 4.5)^2}{\tau_e max} - \frac{U \cdot 1}{4}$$

and the length of the rods to be bent:

$$z_, = \frac{2 P}{m \cdot \sigma_d \cdot \vartheta}$$

where m is the number of rods to be bent and ϑ the diameter of the rod.

To prevent the rods from breaking z should never be greater than $6 . \vartheta$.

C. PRESSURE ON DAM

P_1 = 1C . 62.5 = 3 . 62.5 = 312.5 lbs. per sq. ft.

or: P_1 = la tgd . 62.5

la = $lb^2 + ab^2$

lb = ab = 5 ft.

la = $2(ab^2)$ = ab 2 = 5 2 = 7.071

tgd = lc : la

lc = lb = 5ft.

la = $5\sqrt{2}$

62.5 . tgd = 62.5 : $\sqrt{2}$ = 44.19

Then:

P_2 = (ab + 16) . 44.19 = 21.56 . 44.19 = 967

P_3 = (21.56 + 16) . 44.19 = 37.56 . 44.19 = 1685

P_4 = (37.56 + 16) . 44.19 = 53.56 . 44.19 = 3100

P_5 = (53.56 + 16) . 44.19 = 85.56 . 44.19 = 3100

P_6 = (69.56 + 16) . 44.19 = 85.56 . 44.19 = 3850 lbs. per sq.ft

1 bb/sq. ft. = 5.076 kghn²

P_1 = 315.2 . 5.076 = 1600 kghm²

q = 1600 . 4.58 . 0.01 = 73.3 kghm²

$$M_C = \frac{73.3 \;.\; 2440^2}{8} = 54{,}500{,}000 \text{ cmkg}$$

C = 458 cm.

$$h = \sqrt{\frac{M_C}{C}} = 0.432\sqrt{\frac{5.45 \;.\; 10^6}{458}} = 149 \text{ cm.}$$

d = 0.2 . 149 = 29.8 cm.

d_1 = 11.75 in.

fe = 0.00587 . 149 . 458 = 400 cm²

fe_1 = 62 sq. in.

P_2 = 967 . 5.076 = 4900 kghm²

q = 4900 . 4.58 . 0.01 = 224 kghm²

$$M_C = \frac{224 \;.\; 2440^2}{8} = 166{,}500{,}000 \text{ cmkg.}$$

C = 458 cm.

$$h = 0.432\sqrt{\frac{166.5 \;.\; 10^6}{458}} = 261 \text{ cm.}$$

d_2 = o.2 . 261 = 52.2 cm. = 20.5 in.

fe = 0.00587 . 261 . 458 = 700 cm²

fe_2 = 108 sq. in.

P_3 = 1685 . 5.076 = 8550 kghm²

q = 4.58 . 0.01 . 8550 = 390 kghm²

$$M_C = \frac{390 \;.\; 2440^2}{8} = 290{,}000{,}000 \text{ cmkg.}$$

C = 458 cm.

$$h = 0.432\sqrt{\frac{290. \; 10^6}{458}} = 344 \text{ cm.}$$

d_3 = 0.2 . 344 = 68.8 cm. = 27 in.

fe = 0.00587 . 344 . 458 = 925 cm²

$fe_3 = 143$ sq. in.

$P_4 = 2400 \cdot 5.076 = 12500$ kghm²

$q = 458. \ 0.01 \cdot 12500 = 554$ kghm²

$$M_C = \frac{554 \cdot 2440^2}{8} = 412{,}000{,}000 \text{ cmkg.}$$

$$h = 0.432 \sqrt{\frac{412 \cdot 10^6}{458}} = 410 \text{ cm.}$$

$d = 0.2 \cdot 410 = 82$ cm. $= 30.2$ in.

$fe = 0.00587 \cdot 410 \cdot 458 = 1100$ cm²

$fe_4 = 170$ sq. in.

$P_5 = 310 \cdot 5.076 = 17700$ kghm²

$q_5 = 800$ kghm²

$$M_C = \frac{800 \cdot 2440^2}{8} = 595{,}000{,}000 \text{ cmkg.}$$

$$h = 0.432 \sqrt{\frac{595 \cdot 10^6}{458}} = 494 \text{ cm.}$$

$d = 0.2 \cdot 494 = 98.8$ cm. $= 39$ in.

$fe = 0.00587 \cdot 494 \cdot 458 = 1390$ cm² $= 216$ sq. in.

$P_6 = 5850 \cdot 5.076 = 19500$ kghm²

$q_6 = 4.58 \cdot 0.01 \cdot 19500 = 893$ kghm²

$$h = 0.432 \sqrt{\frac{893 \cdot 2440^2}{458}} = 536 \text{ cm.}$$

$d_6 = 107.2$ cm. $= 43$ in.

$fe_6 = 1500$ cm² $= 230$ sq. in.

STRESSES IN THE BEAMS

The zone of pressure will be at:

$$x = \frac{n \cdot fe \cdot h + \frac{C \cdot d^2}{2}}{C \cdot d + n \cdot fe}$$

and for point 1: $X = 52.6$ cm.

$$h_1 = \frac{2hx - xd - dh + 2/3d^2}{2x - d} = 133.5 \text{ cm.}$$

$$\sigma_e = \frac{M_C}{fe.h_1} = \frac{54.5 \,.\, 106}{400 \,.\, 133.5} = 1000 \text{ kg/m}^2$$

THE COMPRESSIVE STRAIN OF THE CONCRETE

$$\sigma_d = \frac{\sigma_e}{n} - \frac{x}{h - x} = \frac{1000}{15} \,.\, \frac{52.6}{149 - 52.6} = 37.5 \text{ kg/m}^2$$

DETERMINATION OF THE SHEARING FORCE

The reacting pressure on one end of the beam will be:

$$R = \frac{Q}{2} = \frac{g.1}{2} = \frac{73.3}{2} = 2440 = 89500 \text{ kg.}$$

$$C_1 = 98.5 \text{ cm.}$$

$$\tau\text{ max.} = \frac{89500}{133.5 \,.\, 98.5} = \frac{R}{h_1.C_1} = 6.82 \text{ at.}$$

The load not carried by bond would be:

$$T = \frac{(\tau\text{max} - 4.5)^2}{\tau\text{ max}} \,.\, \frac{C_1 \,.\, 1}{4} = 47280 \text{ kg.}$$

Area of the rods to be bent under an angle of 45° at the end to take up this strain:

$$fe' = 0.707 \frac{.\, 0.8\, T}{\sigma_e} = \frac{0.707 \,.\, 0.8.47280}{1000}$$

$$fe' = 26.5 \text{ cm}^2$$

The area of one rod:

$$\int^2 \frac{\pi}{4} = \frac{5{,}02^2 \,.\, \pi}{4} = 20.3 \text{ cm.}^2$$

$$i = \frac{26.5}{20.3} = 1.3 \text{ ; 2 rods are to be bent.}$$

$$z = \frac{2\,P}{m \cdot \sigma_d \cdot \int} \quad \text{where } P = \frac{(\tau max - 4.5)^2}{\tau max} \cdot \frac{U.1}{4} = 6832\text{kg.}$$

$$z = 3.35 \text{ cm.}$$

THE DECK SPANS

For point 1 we have:

$h = d_1 = 32$ cm. $C = 15$ ft. $= 458$ cm.

$h = 30$ cm.

$fe = 0.00603 \cdot C \cdot d_1 = 88.4 \text{ cm}^2$

an evenly distributed pressure.

For point 2 we have:

$fe_2 = 0.00603 \cdot 458 \cdot 55 = 152 \text{ cm}^2$

For point three:

$fe_3 = 0.00603 \cdot 458.72 = 206 \text{ cm}^2$

For point 4:

$fe_4 = 0.00603 \cdot 458 \cdot 82 = 226 \text{ cm}^2$

For point 5:

$fe_5 = 0.00603 \cdot 458 \cdot 98.8 = 272 \text{ cm}^2$

For point 6:

$fe_6 = 0.00603 \cdot 458 \cdot 107 = 296 \text{ cm}^2$

Table 1 gives the figures arrived at in this way and the complete arrangement and dimensions are shown in Fig. 10.

From Table 1 we see there are 43.6 lbs. of steel in each cu. yd. of concrete or about 33%. Allowing 3 cts. per lb. of steel for cost, haulage and placing we have:

43.5 . 3 = $1.30 per cu. yd. for the steel.

Practically the same buildings would be needed as for the construction of the rubble concrete dam. Excavation costs and washing of the sand would be about $35 per day. The mixing

TABLE 1

Point	SPANS: C in cm.	SPANS: d in cm.	SPANS: fe in cm^2	SPANS: Rods	BEAMS: h in cm.	BEAMS: z in cm.	BEAMS: fe in cm^2	BEAMS: Rods	WEIGHTS: V_b	WEIGHTS: V_s	WEIGHTS: W_c	WEIGHTS: W_e	WEIGHTS: W_m	WEIGHTS: W_t
I	458	30	88.4	1x32--1"x60' i=2 z'=1 1/3 cm.	149	3.25	400	2x23 -- 2"x35' z = 2	904 cu. yds. concrete 46--2"x35'+23--2"x45' rods	860 cu. yds. concrete 296--1" x 60' rods	9,370,000 lbs.	77,000 lbs. 43.6 lbs. per cu. yd.	150,000 lbs.	9,597,000 lbs.
2	458	52.2	152	1x40--1"x60' i =2 z=1 1/3"	261	3.25	700	"						
3	458	68.8	206	1x140--1"x60' i =2 z=1 1/3"	344	3.25	925	"						
4	458	82	226	" "	410	3.25	1100	1x23 -- 2"x40' z = 2						
5	458	98.8	272	" "	494	3.25	1390	"						
6	458	107.2	306	" "	536	3.25	1500	"						

plant would be the same as needed for the rubble concrete dam in arrangement and capacity of 130 cu. yds. per day at $0.10 per cu. yd.

The main dam of 1764 cu. yds. of concrete would cost, including labor and plant charges:

	Foremen	Skilled Labor	Labor	Cost cu. yd.
Sand	1	2	8	$1.40
Crushed rock	1	4	4	0.48
Forming	4	20	16	4.46
Mixing	2	2	25	0.89
Placing	4	6	20	1.75
Cement				2.30
Placing of steel				1.30
			Total	$12.68

Lumber for forms will average about $2800 if used in two sections and for the corewalls. The cost of corewalls and earth embankments we assume as above.

Table 2 gives the costs of both constructions from which we find it more economical to put in the reinforced concrete dam.

Items	Concrete Gravity Dam		Reinforced Gravity Dam	
	per day	total	per day	total
Buildings		2735		2735
Gravel	55.80	6100	55.80	530
Stone	44.70	4920	47.20	141
Sand			35.00	105
Mixing	11.42	1250	14.10	430
Steel (in place)			1.30	230
Placing	26.80	2950	19.80	5940
Formers		900		210
Main Dam (per day (cu. yd.	136.72 1.055	18855	173.20	21265
Labor per cu. yd.	5.90	8450	12.68	4940
Corewalls /cu. yd.	6.11	16475	6.11	16475
Earthwork	0.40	2160	0.40	2160
Power house		10000		
Lumber				380
Contingencies		7572		500
Superintendency 6% - 15%		9086		1400
Total		168087		131540

THE TURBINE UNITS

A. The Hydraulic Losses

If a quantity of water falls a distance H the work done is:

$$A = 1000 \cdot Q.H \text{ m/sec.}$$

and the theoretical power

$$N_a = \frac{1000 \cdot Q \cdot H}{75}$$

Part of this will be lost in overcoming the resistance of the water passing thru the intake chamber or pipes and thru eddying. If we express this loss in a fraction of the total head H we would have:

ρH where $\rho < 1$ or

placing $1 - \rho = \varphi$ we would have $\varphi \cdot H$ of the head left for use in the turbine.

Another loss is had when the water leaving the wheel has a certain velocity of expulsion which is necessary to carry the water away from the wheel so that it will not be dragged along after it has given up its energy in passing thru the wheel. If this is expressed by $\delta \cdot H$ we would now have as an actual head $H [1 - \rho - \delta]$

A further loss due to the mechanical friction in the bearings can be expressed by $\mu \cdot H$ which would give us as the available head for utilization:

$$H [1 - \rho - \delta - \mu]$$

Substituting this in the above equation:

$$Ne = \frac{1000 \cdot Q \cdot H [1 - \rho - \delta - \mu]}{75}$$

and the efficiency factor:

$$N = \frac{Ne}{Na} = 1 - \rho - \delta - \mu$$

From hydraulics we know that, the velocity w by a constant head is:

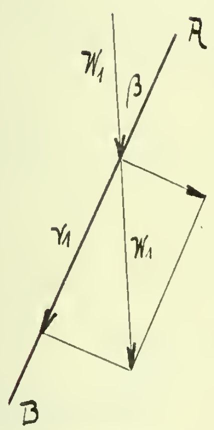
$$w = 2\ g.h$$

squaring this and dividing by 2g we have:

$$\frac{w^2}{2g} = H \quad \text{or}$$

the head is equal to the square of the velocity divided by 2g.

If a continuous stream of water strikes a firm and smooth straight wall AB under an angle B with the velocity W_1 the velocity must resolve itself into two components the one v_1 lying in the



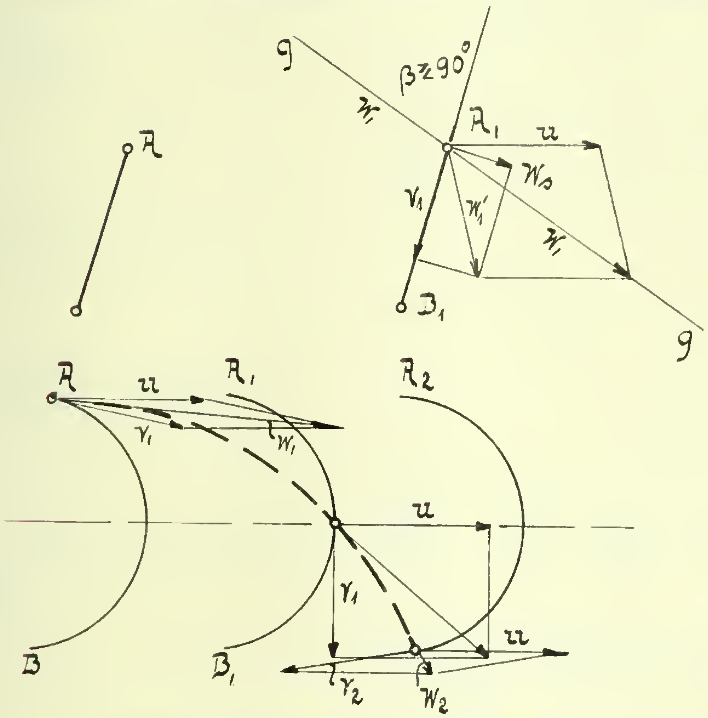
plane of the wall, the other vertical to the wall W_s. The latter we will call the component of shock and is destroyed by the wall. The water will then flow parallel to the wall with the velocity v_1. The energy of the unit of volume (weight) with which the water flows parallel to AB is:

$$\frac{V_1^2}{2g} = \frac{W_1^2}{2g} - \frac{W_s^2}{2g}$$

If the loss $\frac{W_s^2}{2g}$ shall equal zero the angle B must equal zero or the water must strike the wall tangential to the surface.

If the wall AB moves parallel to itself with a velocity w the line g + g would be the path of the water and after a short interval the wall would have the position A'B'. In A' the veloci-

ty V_1 resolves itself in two components the one must equal and have the same direction as u so that the water can follow the wall, the other W_1' must again be resolved into two components one of these V_1 must be in the plane of the wall the other, the component of shock, vertical to the wall.



We will call u the component of transportation, V_1 the relative velocity and W_1 the absolute velocity. From the hydraulics we know that at the most only one half of the energy is transferred to a straight wall and that this is when $B = 90°$ and $u = \frac{W_1}{2}$ and vertical to AB.

Considering a curved wall or surface AB (see the accompanying figure) the water would strike the wall without shock if the two above requirements are fulfilled, namely that W_1 resolves itself into two components, the component of transportation equal to u the velocity of the wall AB and the relative velocity V_1 which must be tangent on the surface AB. If now the water shall flow without shock in the direction of the wall AB, the relative velocity V_1 must be tangent on the wall at every point, e.g., A_1B_1 and A_2B_2. Due to the deflection of the jet by the curved wall the velocity of expulsion W_2 is very much smaller as can be seen

in the accompanying figure, the energy given up by the jet is:

$$a = \frac{w_1^2 - w_2^2}{2g}, \text{ where}$$

$\frac{w_2^2}{2g}$ is the energy with which the water leaves the bucket (wall) and is equal to δ. H. we can therefore write:

$$\frac{w_2^2}{2g} = \delta . H. \text{ for a head of H meters.}$$

If β_1 = 0 we have:

$$w_1 = u + v_1$$

and for B_2 = 180°

$$W_2 = u - v_1; \text{ for } u = v_1$$

$W_2 = 0$ or $W_1 = 2u$ and $u = \frac{W_1}{2}$. For this case then the entire energy:

$$a = \frac{W_1^2}{2g} \text{ is received.}$$

(This is used by Pelton wheels)

If a quantity of water (Q m^3per sec.) flows thru a tube, which converges uniformly toward the end (as in the accompanying figure) under a constant head H meters the velocity of the water at a certain point will be:

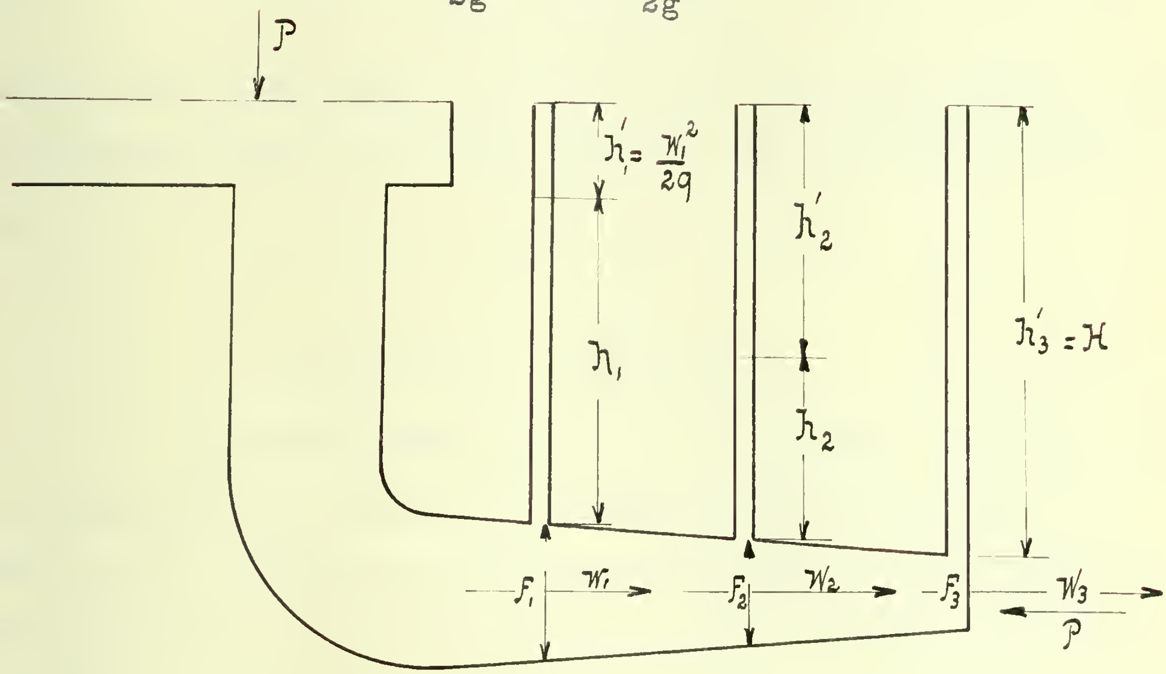
$$w_1 = \frac{Q}{F_1}; \; W_2 = \frac{Q}{F_2} \text{ etc.}$$

or $$w_1 . F_1 = W_2 . F_2 = Q \text{ where}$$

$W_1 < W_2 < W_3$ etc. are the velocities at each section. Each velocity corresponds to a certain head h which at

the same time is the energy of the unit of velocity. We therefore can write:

$$h'_1 = \frac{W_1^2}{2g}\ ;\ h^1_2 = \frac{W_2^2}{2g}\ \text{etc.}$$



This is the kinetic energy of the water in motion for a given section of the tube. By the law of the conservation of energy the same energy must be present in each section of a column of water in motion under constant pressure. As the kinetic energy for each succeeding section increases from left to right, we will call h_1, h_2, h_3, etc., the potential energy which decreases from left to right and we have:

$$\frac{W_1^2}{2g} + h_1 = \frac{W_2^2}{2g} + h_2 = \frac{W_3^2}{2g} + h_3 \ \text{etc.}$$

At point 3 the entire head H will be converted into kinetic energy and $h_3 = 0$ or

$$W_3 = \sqrt{2gH};\ H = \frac{W_3^2}{2g}$$

The pressure, the potential energy in section $F_1 = h_1 = H - \frac{W_1^2}{2g}$

is used to increase the velocity W_1 to W_3. The result of the acceleration toward F_3 is a reaction in the opposite direction. The value of this reacting force for the unit of water (1 kg.) is:

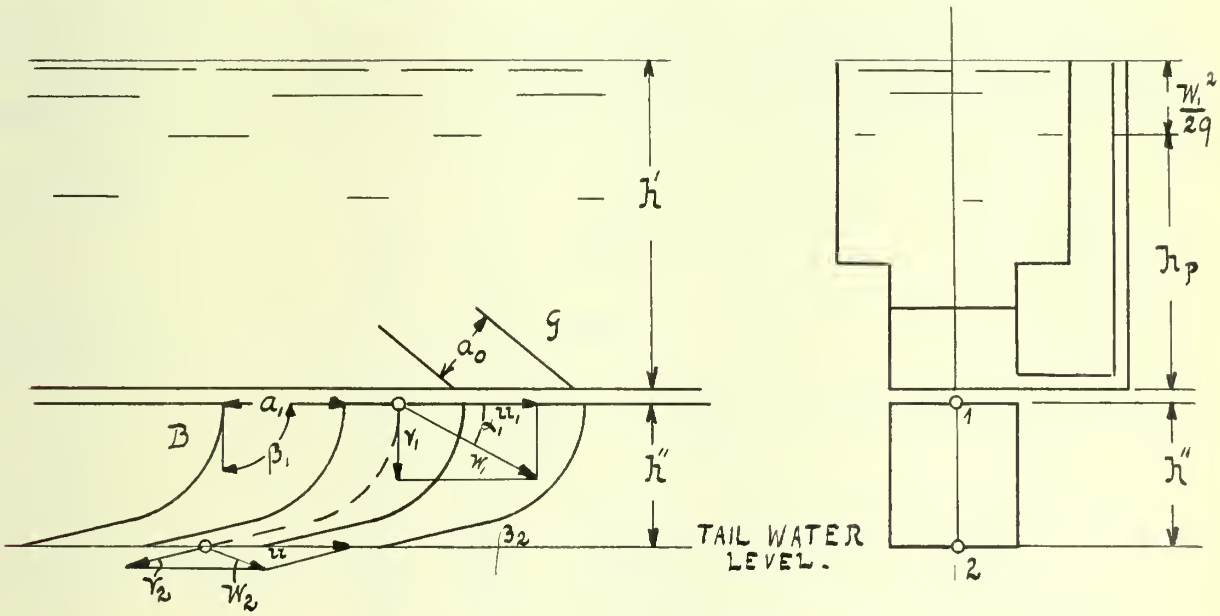
$$r_1 = 1/g(W_3 - W_1)$$

In the figure the water is brought from a vertical position into an horizontal flow, the total reaction for the entire distance s is:

$$r = 1/g \cdot W_3$$

B. THE THEORY OF TURBINES

In the following figure let G represent the gates and B the buckets of a turbine. In considering the action of the water in the turbine we will assume the center of the width of expansion ~~as~~ to lie in the surface of the tailwater at point 2, and will consider what arises in the middle of the water path $\overline{12}$.



From above we know that in order to have the water enter the turbine without shock the absolute velocity W_1 must resolve

itself into two components, the component of transportation u equal to the peripheral velocity of the runner and the relative velocity of the runner and the relative velocity v_1 as tangent to the bucket surface. The water flows, between the buckets with the relative velocity v_1 and leaves the runner at point 2. As the area between the buckets decreases toward point 2 the water will receive an acceleration which in turn will cause reaction. To bring the relative velocity from v_1 to v_2 a certain force is necessary which we will take from the total head, that is, to produce the absolute velocity W_1 we can not use the entire head h' but only a part $\frac{W_1^2}{2g}$ while the remaining hp produces a pressure in the space between gates and runner. We then have the following equation of energy:

$$\frac{W_1^2}{2g} + hp = h'$$

This pressure hp and h" are now used to accelerate the relative velocity v_1 to v_2 and for point 2 we have the following equation of energy:

$$\frac{V_2^2}{2g} - \frac{V_1^2}{2g} = hp + h''$$

Adding these two equations we have:

$$\frac{W_1^2}{2g} - \frac{V_1^2}{2g} + \frac{V_2^2}{2g} + hp = hp + h' + h''$$

$$\frac{W_1^2}{2g} - \frac{V_1^2}{2g} + \frac{V_1^2}{2g} = h' + h'' = H \text{ or}$$

$$W_1^2 - V_1^2 + V_2^2 = 2g \,.\, H.$$

or taking the losses into account:

$$W_1^2 - V_1^2 + V_2^2 = 2g.\,\varphi.\,H$$

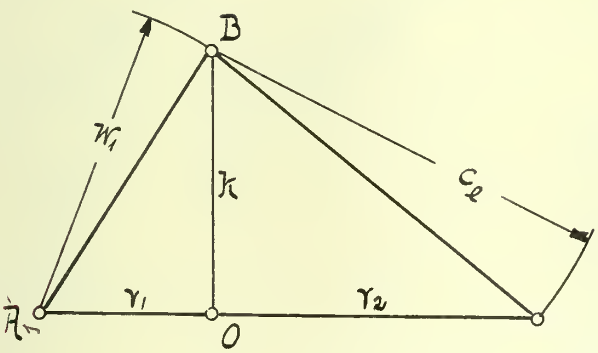
that is the product 2g . φ H. is equal to the square of that velocity which corresponds to the hydraulic effective head φ. H. The pressure h_1 is that part of the hydraulic effective head which in the space between runner and gates has not been transformed into motion. We can write:

$$2g. \varphi . H = ce^2 \text{ or for } H = 1m$$

$ce^2 = 2g. \varphi.$ and the above equations become:

$W_1^2 - V_1^2 + V_2^2 = 2g.\varphi = ce^2$ or for graphical determination we can write it in the form:

$W_1^2 - V_1^2 = ce^2 - V_2^2 = k^2$ where W_1 and ce are the hypothenus and V_1 and V_2 the sides with k as the common leg of two right angle tri-angles, as shown in the accompanying figure.



To receive but one unknown in the above equation certain assumptions must be made. For axsial discharge turbines under normal conditions the angle B_1 at point 1 is made equal to 90°, that is, V_1 vertical to u. We then have:

$W_1^2 = u^2 + v_1^2$ and substituting

$u^2 + v_1^2 - v_1^2 + v_2^2 = 2g\varphi = ce^2$

$u^2 + v_2^2 = 2g\varphi = ce^2$

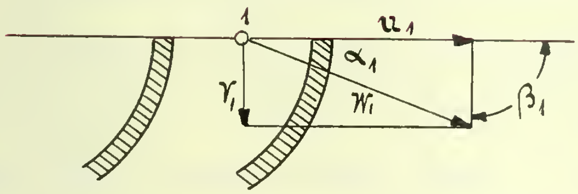
For the point 2 we can also make w_2 vertical to u and receive:

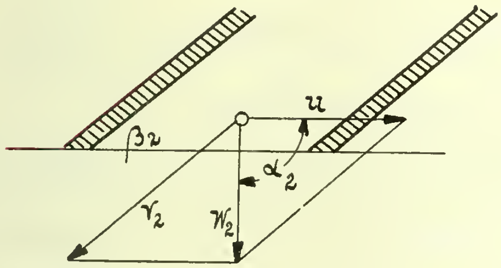
$v_2^2 = u^2 + W_2^2$ for $x_2 = 90°$

and substituting this:

$$u^2 + u^2 + W_2^2 = 2g = ce^2$$

$$2u^2 = 2g\varphi = W_2^2 = ce^2 - W_2^2$$





For H = 1m the loss due to the water leaving the turbine was:

$$\frac{W_2^2}{2g} = \delta \text{ or } W_2^2 = 2g\delta \text{ which}$$

would give:

$$2u^2 = 2g\,(\varphi - \delta) = ce^2 - W_2^2$$

or as $\varphi - \delta = 1 - \rho - \delta$ the hydrl.efficiency:

$$2u^2 = 2g\,.\,n = ce^2 - W_2^2$$

for H = 1m, $B_1 = 4\alpha_2 = 90°$

In the above we have assumed that point 2 lies just on the tail-water level which is of course only a special case. We wish to see what difference it would make if 2 were below or above the tail-water surface. For point 1 we have:

$$\frac{W_1^2}{2g} + hp = h'$$

and for point 2:

$$\frac{V_2^2}{2g} - h = \frac{V_1^2}{2g} + hp + h'' + h$$

adding:

$$\frac{W_1^2}{2g} - \frac{V_1^2}{2g} + \frac{V_2^2}{2g} = h' + h'' = H \text{ or}$$

$$W_1^2 - V_1^2 + V_2^2 = 2gH$$ not considering the losses

or:

$$W_1^2 - V_1^2 + V_2^2 = 2g\varphi . H$$ considering the losses;

it is therefore immaterial whether the runner is placed on the tail-water level or below the same. In the same way if the runner is situated above the tail-water level part of the total head H will act as a pressure, the remaining as a vacuum. In practice the runner is placed at least 1m below the top water level and not more than 7m above tail-water level. In the above discussion a wheel with infinite diameter was assumed and in the following will extend the above conclusions to axial wheels.

C. INFLUENCE OF THE PERIPHERAL VELOCITIES BY RADIAL TURBINES

In modern practice the radial reaction turbines with inward flow first designed by the American Francis are used in preference to the French- Tourneyrow turbines due to the unfavorable arrangement of the gates of the latter and the better and easier regulation of the former.

As the wheel velocity u_2 at the point of expulsion is much smaller than u_1 at the point of entrance the water in the buckets must be subject to the influence of the centrifugal force. If we take a cylindrical vessel filled with water and let it rotate with a constant velocity the water surface will form a parabola caused by the centrifugal force. If we take a particle of water 1 (see the accompanying figure) at a distance x from the axis of rotation having the mass m and peripheral velocity u_x two forces act on the particle mainly the centrifugal force c and

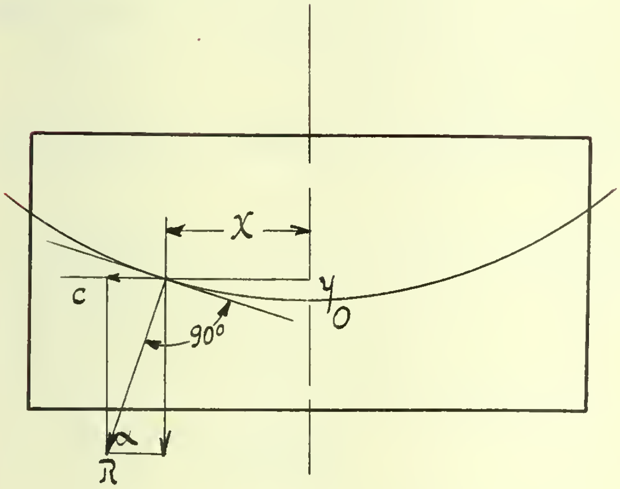
gravity m.g. and to have equilibrium resultant force R must be vertical on the water surface. The centrifugal force for the particle 1 is:

$$c = \frac{m \cdot u_x^2}{x}$$ and if w is the angular velocity:

$$u_x = w.x \text{ or}$$

$$C = \frac{m.w^2.x^2}{x} = m.w^2.x$$

$$tg\alpha = \frac{m.g.}{C} \text{ or } tg\alpha = \frac{M.g.}{m.w^2.x} = \frac{g}{w^2x}$$



We can also put

$$tg\alpha = \frac{dx}{dy} \text{ and}$$

$$g.dy = w^2.\ xdx$$

Integrating this between any two limits we have the pressure between two neighboring particles of water due to the centrifugal force, e.g. from the point O in the axis of rotation to x = 1cm:

$$g.y = w^2 \int_0^x xdx = \frac{w^2x^2}{2}$$

$$wx = u_x$$

$$g.y = u_x^2 \text{ and } y = \frac{u_x^2}{2g}$$

For the points 1 and 2:

$$gy = w^2 \int_{x_2}^{x_1} xdx = \frac{w^2.x^2}{2} - \frac{w^2x_2^x}{2}$$

and as $wx_1 = u_1$ and $wx_2 = u_2$

$$gy = \frac{u_1^2}{2} - \frac{u_2^2}{2}$$

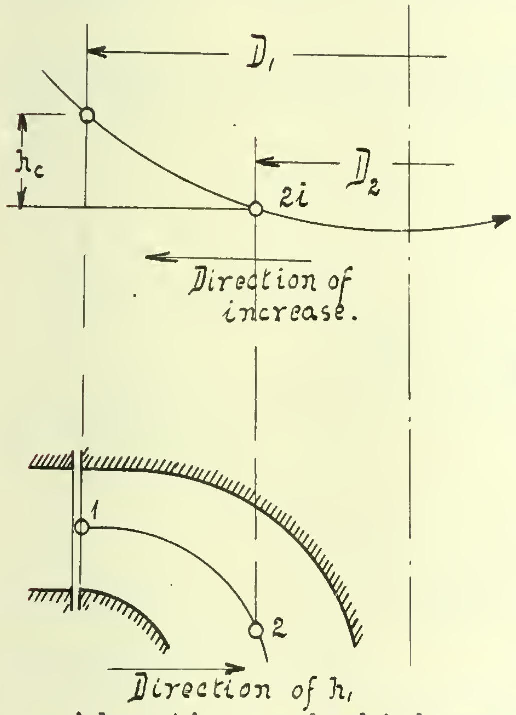
$$y = \frac{u_1^2}{2g} - \frac{u_2^2}{2g} = hc$$

The above results can be applied to the canals or

buckets of a turbine as well. Assume the water enters the runner in 1 and leaves it at 2i in the accompanying figure. The water passing into the runner causes a pressure h_1 to arise which must decrease from 1 to 2i because it increases the relative velocity and at 2i h_1 will have been reduced to zero. The resultant pressure would therefore be:

$$h_1' = h_1 - hg$$

which can be used to increase the acceleration of v_1 to v_2:



$h_1' = \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$ and substituting this in the above:

$$\frac{v_2^2}{2g} - \frac{v_1^2}{2g} = h_1 - \left(\frac{u_1^2}{2g} - \frac{u_2^2}{2g}\right)$$

$$h_1 = \frac{v_2^2 - v_1^2}{2g} + \frac{v_1^2 - u_2^2}{2g}$$

We found that for 1m head:

$$\frac{W_1^2}{2g} + h_1 = \varphi$$

by introducing the value for h_1 into this equation we have an equation which takes the centrifugal force into consideration and which can be directly applied to the turbine:

$$\frac{W_1^2}{2g} - \frac{V_1^2}{2g} + \frac{V_2^2}{2g} + \frac{u_1^2}{2g} - \frac{u_2^2}{2g} = \varphi \text{ or}$$

$$W_1^2 - V_1^2 + u_1^2 - u_2^2 = 2g\varphi = ce^2 \text{ for 1m head.}$$

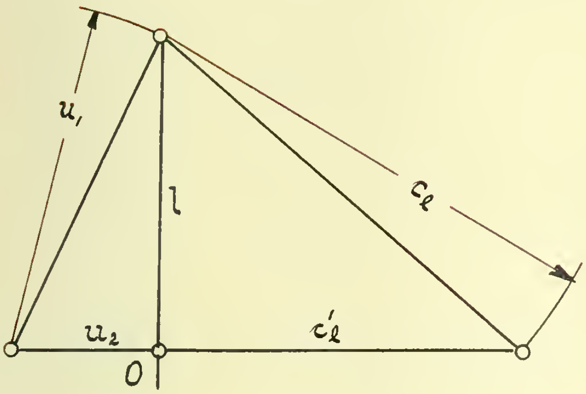
This can be simplified by letting

$$W_1^2 - v_1^2 + v_2^2 = ce^2 - (u^2 - u^2)$$

and $$ce^2 - (u_1^2 - u_2^2) = ce^2;$$

then $W_1^2 - v_1^2 + v_2^2 = ce^2$

$$u_1^2 - u_2^2 = ce^2 - c'e^2 = l^2$$



This equation is shown graphically in the accompanying figure. To note the effect of different gate openings certain assumptions must be made.

1. That $B_1 = 90°$ at point 1 or $v_1 \perp v_1$. We then receive:

$$W_1^2 = u^2 + v^2 \text{ and}$$

$$u_1^2 + v_1^2 - v_1^2 + v_2^2 + u_1^2 - u_2^2 = 2g\varphi$$

$$2u_1^2 + v_2^2 - u_2^2 = 2g\varphi = c_e^2$$

2. That at the point of expulsion $v_2 u_2$ and the former equation would have the form:

$$2u_1^2 = 2g.\varphi = c_e^2$$

For $B_1 = 90°$, $\varphi = 0.87$ and for H = 1m we receive the normal peripheral velocity:

$$u_1 \text{ normal} = \sqrt{g\varphi} = 2.92\text{m.}$$ and for a head of H meters:

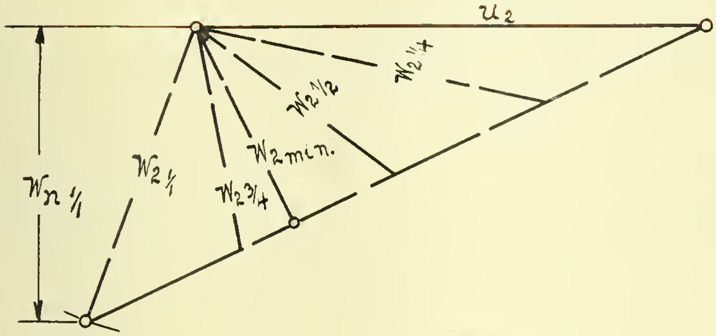
$$u_{1n} = 2.92\sqrt{H}$$

The velocities W_1, v_1 and v_2 will change with every change of gate opening; u_1 and u_2 are functions of H only and will not vary by a constant given head. We would then have:

$$W^2_{1\lambda} - V^2_{1\lambda} + V^2_{2\lambda} = c'e^2 \text{ where}$$

λ is the per cent of gate opening. By $\lambda = 3/4$ several advantages are to be had and for a given gate opening $v_2 = u_2$. We then

have:



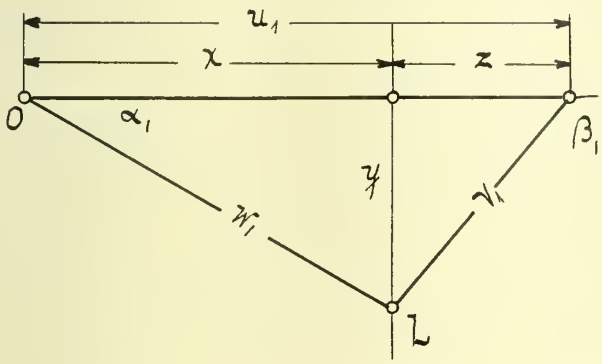
$v_{2(3/4)} = u_2$

$v_{2(1/1)} = 4/3u_2 = 1.33u_2$

In the accompanying figure the graphical determination of W_2 is shown for different gate openings.

In discussing the conditions under which the water enters the receiver by different gate openings we will let the angle B_1 be greater or smaller than 90°; then

$$W_1^2 - x^2 = y^2 = v_1^2 - z^2$$



$W_1^2 - x^2 = v_1^2 - z^2$

$W_1^2 - v_1^2 = x^2 - z^2 = (x + z)$

$X(x + z] = u_1;\ x - z = x -$

$(u_1 - x) = 2x - u_1$ and

$w_1^2 - v_1^2 = u_1(2x - u_1) =$

$2u_1x - u_1^2$

Bringing this into the equation:

$$W_1^2 - v_1^2 + v_2^2 + u_1^2 - u_2^2 = ce^2 \quad \text{we have}$$

$$2u_1^2x - u_1^2 + u_1^2 + v_2^2 + u_1^2 = ce^2$$

or $$2u_1x + v_2^2 = ce^2 + u_2^2$$

For a given wheel the values ce, u_1 and u_2 do not vary with the gate opening but are constant; v_2 varies proportional to the gate opening; therefore x must decrease the greater the gate opening is and increase the less the gates are opened. The

general form of the above equation would be:

$$2u_1x\lambda + v_2^2\lambda = ce^2 + u_2^2$$

If the profit of the wheel is given and u_1 is known:

$$u_2 = u_1 \frac{D_2}{D_1}$$

where D_1 is the outer and D_2 the inner diameter of the buckets.

By different gate openings the vertical L will change its position because x changes proportional to the gate opening. With this the direction and value of the absolute velocity W_1 is changed; this change in the direction of W_1 is brought about by the change in the position of the gates themselves. In the following we will determine the equation of that curve on which the point E moves by different gate openings.

The normal component of W_1, y is vetical to u_1 while u_1 is tangent on the cylindrical area of entrance, therefore y is vertical to the area of entrance. As the cylindrical area of entrance remains constant for all gate openings the velocity or the radial component y must vary proportional to the gate opening and the ratio:

$$\frac{y}{v_2} = \text{constance}$$

$v_2 = \frac{y}{c}$ and substituting in the equation:

$2u_1x + v_2^2 = ce^2 + u_2^2$ we have:

$$2u_1x + \frac{y^2}{c^2} = ce^2 + u_2^2$$

$$c^2 .\ 2u_1x + y^2 = (ce^2 + u_2^2)c^2$$

where x and y are the only variables

$$c^2(ce^2 + u_2^2) = c$$

$$c^2 .\ 2u_1 = 2p = c$$

$$y^2 = c - 2px$$

This is the equation of a parabola for the sonstruction of which u_2 must be determined from the equation:

$$u_2 = u_1 \frac{D_2}{D_1}$$

and v_2 from the equation:

$$v_{2(1/1)} = 4/3u_2$$

The radial component y for any gate opening can be determined for a given runner profit from the equation:

$$Q = \pi D_1 . C_1 . y$$

$$y = \frac{Q}{\pi . D_1 C_1}$$ where Q is the quantity of water flowing thru the turbine. The construction of this parabola is shown in figure 4 Plate 1 of the appendix for definite conditions. Also the graphical determination of the velocities for given gate openings is shown in figure 4 plate 1 which is done in the following manner.

Assume the coordinates A-B, to the right of O the entrance conditions and to the left of O the conditions of expulsion.

1/ the conditions of entrance:

Construct the parabola of entrance with the help of x_o, y_o, $x_{(1/1)}$, $y_{(1/1)}$ and u_1.

$$x_o = \frac{c^2e + u_2^2}{2u_1}, \quad y_o = o, \quad x_{(1/1)} = \frac{c^2e + u_2^2 - v_2^2}{2u_1}$$

and determine the corners of the triangles for the four characteristic gate oprnings:

$$\lambda = (1/1);\ 3/4,\ 1/2,\ 1/4 .$$

By carrying the scaler value of u_1 to the right of O the angle

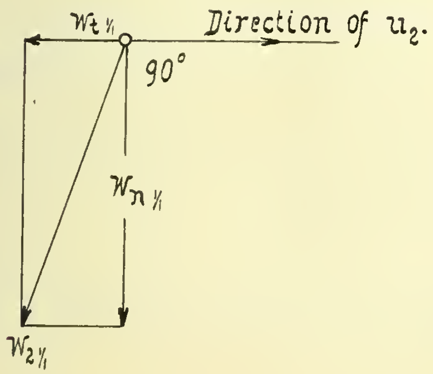
for different gate openings can be determined. We also see that angle B_1 has a different value for each gate opening. This is for practical purposes impossible due to difficult and uneconomical construction and a value must be definitely set for B_1 for all gate openings. We will select a value for B_1 which will give equally good results for 1/2 or for full gate opening and this is almost without exception the case when the most efficient value is given B_1 at 3/4 gate opening. Thru this angle the direction of the buckets is given at the point 1. From the diagram we see that the relative velocity v_1 is tangent to the bucket for only one gate opening and that by all other gate openings there will be a component of shock u_s the value of which can be taken directly from the diagram.

2/ the conditions of expulsion:

By a given u_2 the relative velocity of expulsion can be determined by the equation:

$$v_{2(1/1)} = 1.33\ u_2$$

From the diagram, however, we find that for full gate opening, as $w_{2(1/1)}$ is not vertical on u_2, u_2 is not normal to the plain of expulsion. We must have the normal component of $w_{2(1/1)}$, $W_{n(1/1)}$ (see figure). As $W_{2(1/1)}$ is approximately equal to $W_{n(1/1)}$ we will assume the normal loss of expulsion to be:



$$\delta_{n(1/1)} = \frac{W^2_{n(1/1)}}{2g}$$

The true value being:

$$\delta_{(1/1)} = \frac{W^2_{2(1/1)}}{2g}$$ and is a

little larger than the above. The second component of W_2 which has the same direction of u_2, $w_t(1/1)$ or tangential component has the tendency to cause eddying especially by small gate openings. $\sqrt{}_n(1/1)$ can be assigned equal to 7%. The construction of the diagram of expulsion can be seen from figure.

Returning to the conditions of entrance figure 4 we see that the absolute entrance velocity W_1 steadily increases the smaller gate opening while the pressure h_1 decreases. The velocity W_1 must reach a maximum which we can determine by the following: for any gate opening we have the equation:

$$W_1^2 \lambda - v_1^2 \lambda + v_2^2 \lambda = c'^2e$$

where W_1 increases the greater the ratio $\frac{v_1}{v_2}$ is. A limit would be when $v_1 = v_2$ when $h_1 = o$. That the water in the buckets should be brought to a stop is impossible $(v_1 > v_2)$ and we therefore have a limit:

$$v_1 = v_2$$

$$W_1 max = c'e$$

To bring c'e into the diagram we must use the equation:

$$u_1^2 - u_2^2 = c^2e - c'^2e = 1^2$$

and express this graphically. The entrance curve will be a parabola as far as the intersection of the parabolic curve and the circle with the radius c'e at the point λc and from there on follows the arc of the circle. The shock components by the different gate openings for $B_1 = 90°$ are:

$$h_{s\lambda} = \frac{u^2 s\lambda}{2g}$$

which must be subtracted from the total efficiency and we have:

$$\eta_a = 1 - \rho - \delta_\eta - \frac{us\,\lambda}{2g} - \mu$$

THE PERIPHERAL VELOCITIES OF REACTION TURBINES AND THEIR LIMITS

The minimum value of u_1 can be determined by the following analytic expression:

$$u_1 min = ce - \sqrt{\frac{\cos\alpha(1/4)\sqrt{\cos^2(3/4) - [1-(\frac{D_2}{D_1})^2 - [1-(\frac{D_2}{D_1})^2]}}{2\cos\alpha(3/4)[1-(\frac{D_2}{D_1})^2]}}$$

In practical construction $u_1$min is not taken lower than 2.4 ÷ 2.5m for H = 1m

$u_1max = 3.5 \div 3.8m$ for H = 1m & $B_1 > 90°$

$u_1norm = 2.9 \div 2.92m$ for H = 1m & $B_1 = 90°$

It is perhaps well to add that the equation

$$v_{2(1/1)} = 1.33\ u_2$$

holds only as long as $v_{2(1/1)}$ is equal or smaller than c'_e. To determine an equation which will hold for other values we will again assume a turbine with infinite diameter: $\frac{D_2}{D_1} = 1$ and $u_1 = u_2$ and to apply the following to a radial turbine c'_e must only be substituted for c_e. We had the fundamental equation:

$$W_1^2 - v_1^2 + v_2^2 = c^2e$$

which would give for $u_{1max} = 3.8 = u_2$ in the equation

$$v_2 = 1.33 \ .\ u_2 = 5.07m.$$

For u_1max, $\varphi = 0.84$, therefore

$c^2{}_e = 2g\varphi = 16.5$ and substituting this above:

$$W_1^2 - v_1^2 = c^2e - v_2 = 9.2 \text{ or } v_1 > w_1$$

This would necessitate very large angles B_1 and α_1 which would lead to deficient and difficult constructions. By assuming $v1(1/4) = W_{1(1/4)}$ we have:

$$x_{(1/1)} = \frac{u_1}{2}$$

and $$v_{2(1/1)} = ce \text{ or for radial turbines:}$$

$$v_{2(1/1)} = c'e$$

The above results can be generalized and in determining our units the method used will be plainly shown.

To have our turbine correspond with turbines on the market we will take a Sampson size 17B turbine for which:

$$Q_{(1/1)} = 1.06 \text{ m}^3 \text{ per sec.}$$

$$Q_1 = \frac{1.06}{\sqrt{17}} = 0.257 \text{ m}^3 \text{ per sec.}$$

Assuming the peripheral velocity u_1 for 1 m head to be 3.25m from figure 10 we have:

$$n_1 = \frac{60}{\pi} \cdot \frac{u_1}{D_1} = 146 \text{ r. p. m.}$$

where D_1 = 0.425m. and

$$n = 146\sqrt{H} = 600 \text{ r. p. m.}$$

For $u_1 = 3.25$ $\varphi = 0.863$ as taken from the curve figure 10.

If Co is the height of the entrance canal (see Fig. 1, plate 1) the ratio $\frac{Co}{D_1}$ will give us a direct value for the quantity of water passing thru the turbine. If $y_{(1/1)}$ is the radial component at the point of entrance for full gate opening:

$$Q_1 = \pi D_1 \cdot Co \cdot y_{(1/1)} \text{ or}$$

$$Q_1 = \pi D_1^2 \cdot \frac{Co}{D_1} \cdot y_{(1/1)}$$

and the water passing thru the turbine is proportional to the square of the diameter of the wheel D_1. In construction the limits are:

$$\frac{Co}{D_1} = 0.05 \div 0.5$$

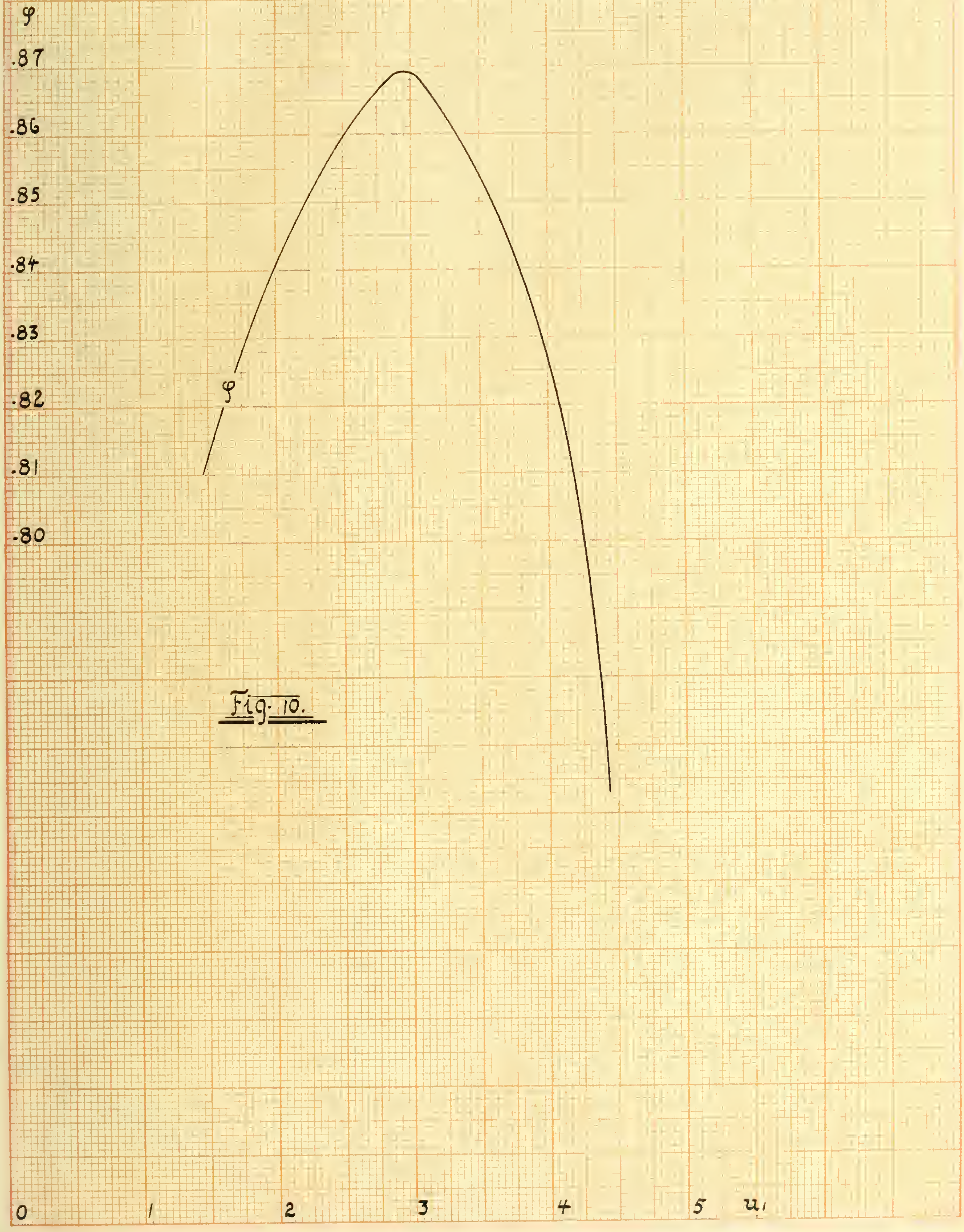


Fig. 10.

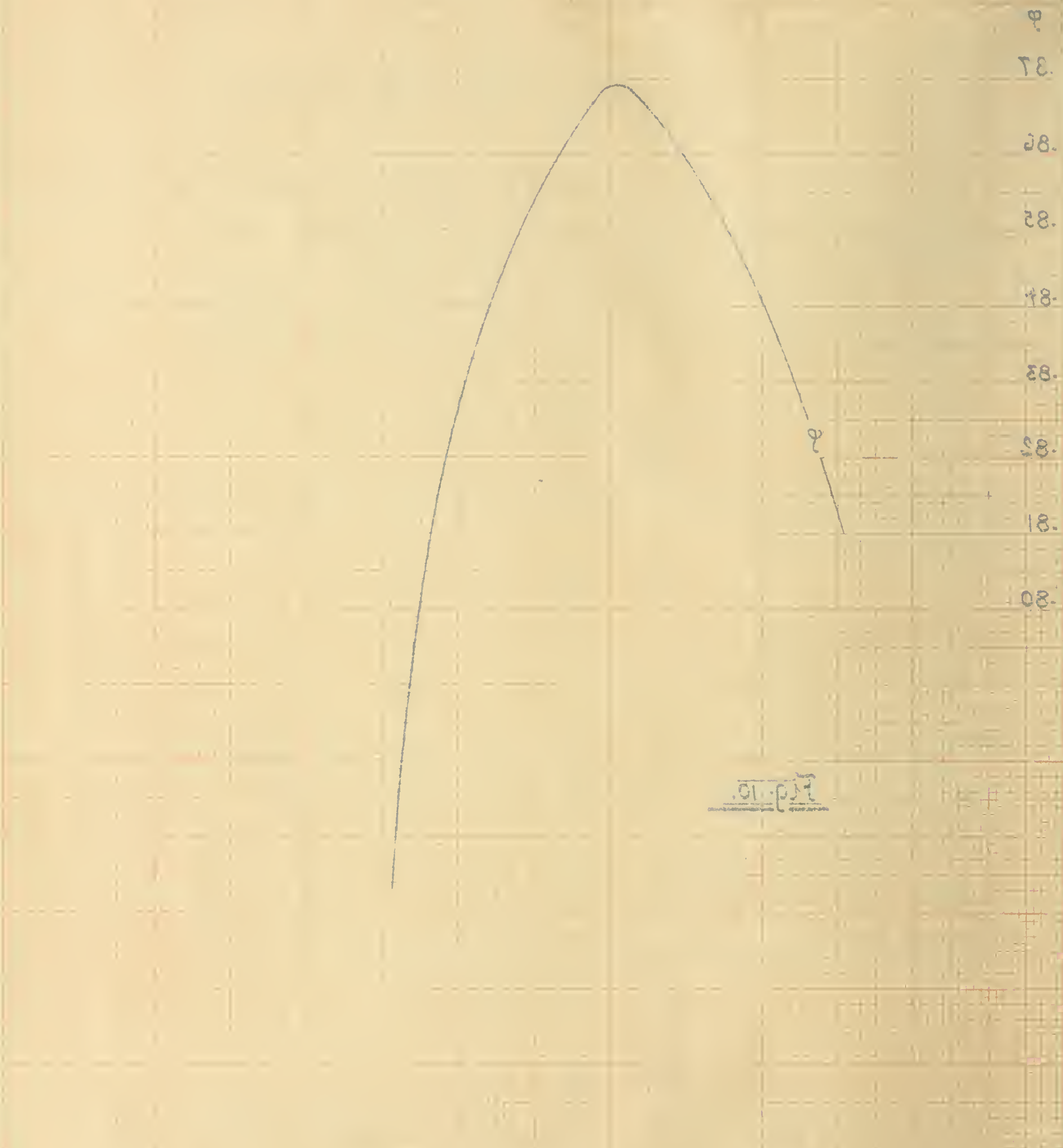


Fig. 10.

If we let $\frac{C_o}{D_1} = S_\gamma$ the relative amount of water passing thru the turbine, as the ratio for the normal type of turbine is $\frac{C_o}{D_1} = 0.2$, we can write for a wheel consuming any absolute quantity of water Q_1:

$$Q_1 = S_\gamma . Q_1 \text{ normal}$$

$$Q_{1n} = 0.75 . D_1^2$$

$$Q_1 = S \quad . 0.75 D_1^2 \text{ or}$$

$$S_\gamma = \frac{Q_1}{0.75 D_1^2}$$

For our wheel:

$$S_\gamma = \frac{0.257}{0.75 . 0.425^2} = 1.9$$

From the curves of Fig. 10a, we find for

$$S_\gamma = 1.9 \text{ , } \frac{C_o}{D_1} = 0.35 \text{ or:}$$

$$C_o = 0.35 . 0.425 = 0.149\text{m.}$$

As v_2 the relative velocity of expulsion varies proportional to the gate opening we could let the diameter of the center of average flow D_{2m} equal $\frac{D_{2a} + D_{2i}}{2}$ with very small error. In our design, however, it is better to take $W_n(1/1) = 1.45$ for $\frac{C_o}{D_1} = 0.35$ and:

$$\frac{D_{2m}}{D_1} = 0.756 \text{ from Fig. 10a}$$

then

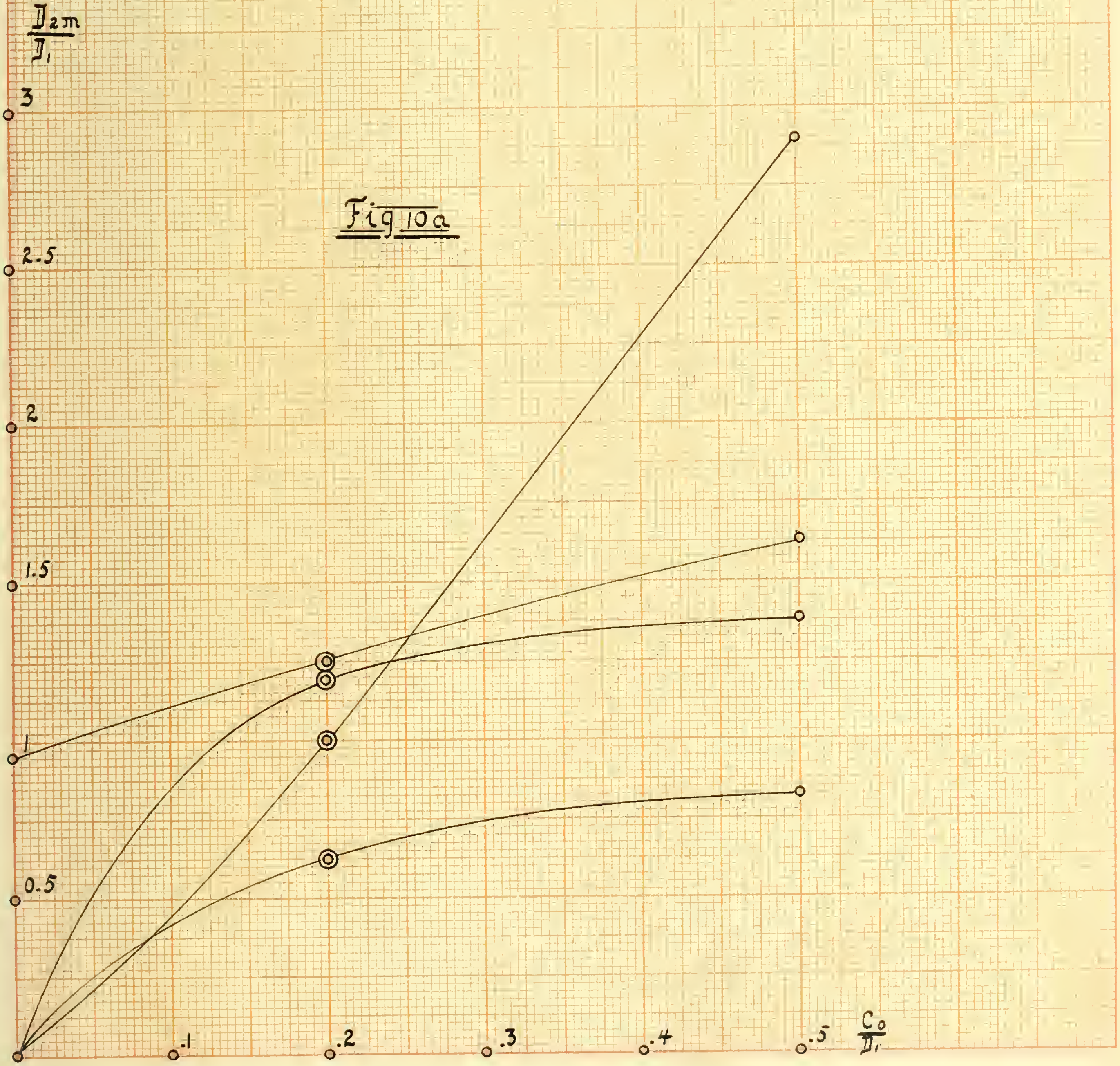
$$D_1 . 0.756 = D_{2m} = 0.321\text{m.}$$

The point at which the expulsion takes place can be determined by assuming:

$$W_3(1/1) = c_3 . W_n(1/1)$$

where c_3 is a factor of correction varying with the speed, for our wheel equal to 0.8. The area of the surface of expulsion is:

Fig 10a



$$F_3 = \frac{\pi \cdot D_3^2}{4}$$

and the quantity of water passing thru it:

$$Q_1 = F_3 \cdot W_3(1/1) = \frac{\pi D_3^2}{4} \cdot C_3 \cdot W_n(1/1)$$

and $$D_3 = \sqrt{\frac{4Q_1}{\pi C_3 \cdot W_n(1/2)}} = \sqrt{\frac{4.\ 0.257}{\pi \cdot 0.8\ .\ 1.45}} = 54.25 cm$$

A vertical section of the wheel can now be drawn and to follow the flow of the water in the wheel more closely we will divide the turbine into four partial turbines each having the same intake and discharge areas. To do this we divide Co into four equal parts; for the first quarter of the discharge area we have:

$$\frac{\pi \cdot D_{3a}^2}{4} = 1/4\ \frac{\pi D_3^2}{4}$$

$$D_{3a} = 1/2\ D_3 = 27.12\ cm.$$

for the second quarter:

$$\frac{\pi \cdot D_{3C}^2}{4} - \frac{\pi \cdot D_{3a}^2}{4} = 1/4\ \frac{\pi D_3^2}{4}$$

$$D_{3C} = 38.4\ cm.$$

and $D_{3c} = 47.$ cm.

An imaginary line AB Fig. 1, plate 1, can be drawn and the points 1, 2, 3, and 4 determined on the circles with the diameter D_{3a}, D_{3C}, D_{3c}, thru which the boundaries of the partial-turbines 1a-2a must pass. This and the line of expulsion 2i-2a must be assumed for the present and corrected later. To construct the diagram of velocity Fig. 2, project 2i ÷ 2a from Fig. 1 on an assumed axis and determine 2m at D_{2m} = 0.321m. By velocities above 2.9 the best results have been attained by letting $C'e_m = u_1$ the equation:

$$V_{2m}(1/1) = c'e_m = \sqrt{c^2e - (u_1^2 - u_m^2)}$$

has the form:

$$V_{2m(1/1)} = c'e_m = u_1$$

Furthermore:

$$2u_1 \cdot x_\lambda + v^2_{2m(\lambda)} = c^2e + u^2_{2m}$$

and from this the coordinates of the entrance parabola:

$$xo = \frac{c^2e + u^2_{2m}}{2u_1} = 3.7$$

$$yo = 0$$

and the coordinates for the gate opening $\lambda = 1/1$ (full gate opening):

$$x1/1 = \frac{c^2e + u^2_{2m} - v^2_{2m}}{2u1} = 2$$

$$y_{(1/1)} = \frac{Q_1}{D_1 . Co} = 0.129 \text{ for } H = 1m$$

The line drawn from u_1 to 0 as in Figure 2 will give graphically the values of u_2 for any point on 2i ÷ 2a. Because of the high ratio $\frac{Co}{D_1}$ we will assume the normal component of $W_{2(1/1)}$, $Wn1/1$ for every point of the line of expulsion to be the same. This will cause the width of expulsion (space between buckets) to vary.

By normal wheels where $u1 = 2.9$ and $S_\Upsilon = 1$ the graphical determination of v_2 by projecting the points 2a, 2c, 2b, 2a and 2i on the axis 0 is fairly exact. A more exact determination especially so for higher speeds is had by dividing the projection of 2a ÷ 2i into four equal parts and determining the velocities as shown in Figure 2. As the working pressure shall not exceed (0.15 ÷ o.25)H for $1m^2$ of bucket surface we can write:

$$F \cdot z = \frac{Q_1 \cdot n}{h \cdot u_{2m}}$$

where F is the projection of the bucket section on the axis 0, n the efficiency, h = 0.2H and u_{2m} the velocity of point 2m. The number of buckets is therefore:

$$z = 10$$

The thickness of the buckets:

$$s_2^1 = 10 \; . \; D\sqrt{\frac{H}{z}} \; + \; 2 \; = 6.25 \text{ mm.}$$

In calculating the quantity of water leaving each partial turbine we can not take the width of expulsion as the distance 2a+2c in Figure 1, as the jet does not cut the line of expulsion at right angles. The diameter of the circle Ab_2 which is tangent to both 1c + 2c and 1a + 2a and whose center is on the line of expulsion must be taken. The length of the partial jet is not the distance between the buckets but a line vertical to the buckets a_2^1. The area of the partial jet is then:

$\Delta f_2 = a_2^! \; . \; \Delta b_2$ and the quantity of water:

$$\Delta q_2 = \Delta f_2 \; . \; v_2$$

The thickness of the buckets must also be considered and we then have instead of $a_2^!$, $a_2^! + s_2^!$. The graphical determination of these values for $a_2^! + s_2^!$ for the points of discharge is shown in Figure 3 plate 1. The determination of the buckets is shown on plate 2 where the method used can be easily followed. Plate 3 shows the gate positions for full gate opening. From data collected by Professor E. H. Waldo of the E. E. Dept. of the U. of Illinois, the cost per h. p. of efficient turbines is approximately:

$$c = 6.6 + 120H^{-.9}$$

Our units would cost approximately $3000. The exciter unit would cost about $200. Including hauling and installation the turbine units would cost $10200.

THE ELECTRICAL FEATURES

The Hoot Lake transmission line runs 2 1/2 miles south of Fergus Falls directly to Dayton Hollow and from here westward to Breckenridge, which would not require a transmission line to connect in the new station. The transformer and high tension disconnecting switches would be placed in a separate building. The Hoot Lake line is fed by G. E. water cooled transformers, Y connected on the primary and Δ connected on the secondary delivering 38,100 volts at 110 switch board voltage.

Clinton and Hancock are to be added to the system, the former having a maximum demand of 42 K. W. and the latter 47 K.W. as taken from the assumed load curve. This will necessitate 15 miles of transmission line and distributing systems for the two towns. Additions to the distributing systems in some of the towns must also be made.

The present installations in the power station will be:

1 Turbine-Generator unit

1 Turbine-Generator exciter unit

1 Induction Motor-Generator exciter set

3 single phase transformers

3 single pole high-tension disconnecting switches

Switchboards and equipment

Lightning arresters

From the power station to the transformers in a separate building the power will be transmitted in cables passing thru the abutments of the dam.

RATING

As the demand for power has increased very rapidly in the last few years it is estimated that after two years the second turbine-generator set will have to be installed and after five years the station will be working at its full capacity as stated on page 13. For the first two years the single set will carry the peak-load, from about 4 p. m. until 7 p. m. for the first year and from 3 p. m. until 8 p. m. for the second year, giving us approximately 450,000 and 750,000 K.W.H. output respectively.

At the end of the second year another set is to be added taking the peak-load from 4 p. m. until 7 p. m. and the output of the station for the third year will be approximately 1,200,000 K. W. hrs. increasing until the end of the fifth year when the third set must be installed, the plant then having a total output of approximately 5,000,000 K. W. hrs.

The capital investment for the first year including cost of installation would be:

Item	Total value
Organization	$10,000
Land	3,500
Dam	131,540
Transformer building	2,000
Turbine	4,000
Generator	9,530
Exciter set	1,400
Accessory power equipment	1,500
Transformers	4,600

Item	Total value
Switchboards	$3,000
Motor-Generator set	1,110
Transformer Equipment	1,500
Miscellaneous Electrical Equipment	250
Engineering and Superintendence	2,200
Tools and Implements	500
Transmission System	16,600
Distribution System	2,000
Line Transformers and Devices	1,200
Electrical Meters and Installation	2,000
Municipal Street Lighting	600
Substation Building	1,000
Miscellaneous	500
Total	$200,530

The operating expenses of the first year would be:

Item	Total value
Station labor and superintendence	$1,700
Supplies and Station Expenses	500
Repairs of Building	500
Repairs of Turbine	75
Repairs of Electrical Equipment	700
Miscellaneous Station Repairs	500
Transmission and pole Repairs	1,000
Substation Labor	1,000
Substation Supplies and Expenses	250

Item	Total value
Repairs of Substation Building	$ 100
Repairs of Substation Equipment	100
Electric distribution superintendence	960
Records and expenses	500
Distribution pole and fixture repairs	800
Repairs of services	1,000
Repairs of transformers	400
Repairs of meters and operation	100
Installation expenses	700
Installation of incandescent operation	900
Installation of incandescent repairs	250
Commercial administration	1,800
Officers and clerks	2,100
Office supplies and expenses	650
Miscellaneous general expenses	1,000
Total	$11,585

During the second year the following investments will be made:

Distribution System	1,000
Line transformers and Devices	600
Electric meters and installations	2,000
Municipal street lighting	800
Miscellaneous electrical equipment	250
Total	$4,650

The cost of operation will increase about 10% making the yearly total $19,360.

At the beginning of the third year the installation of the second turbine-generator set will bring in the following additional expenses:

Item	Total value
Turbine	$4,000
Generator	9,530
Distribution System	1,500
Superintendence and labor	960
Distribution system	500
Line transformers and devices	300
Electric meters and installation	4,500
Total	$21,290

The cost of operation will increase about 25% making the yearly total $24,200.

The following are the investments for the fourth year:

Item	Total value
Distribution system	$ 500
Electric meters installation	2,200
Miscellaneous electrical equipment	200
Total	$ 2,900

The increase in plant operation will be about 15% or a yearly cost of $28,900.

At the beginning of the fifth year the third turbine-generator set is to be added and the total investment would be:

Table 1

Item	Total value
Organization	$10,000
Land	3,500
Dam	131,540
Transformer building	2,000
Turbines	12,000
Generators	28,600
Exciter set	1,400
Motor-Generator set	1,110
Accessory power equipment	1,500
Transformers	4,600
Transformers equipment	1,500
Switchboards	7,300
Miscellaneous electrical equipment	1,500
Engineering and superintendence 10%	12,300
Tools and implements	500
Transmission system	16,700
Distribution system	5,000
Line Transformers and Devices	3,000
Electric meters and installation	15,000
Municipal street lighting	1400
Substation building	1,000
Miscellaneous	800
Total	$268,250

The operating expense for the fifth year would be:

Item	Total value
Station superintendence and labor	$3,780
Supplies and station expense	1,000
Repairs of building	500
Repairs of Turbines	250
Repairs of electrical equipment	1,000
Miscellaneous station repairs	500
Transmission pole repairs	500
Transmission system repairs	1,000
Substation labor	960
Supplies and expenses	250
Repairs of substation building	100
Repairs of substation equipment	100
Electric Distribution superintendence	1,500
Records and expenses	500
Distribution pole and fixture repairs	2,000
Repairs of services	2,500
Repairs of transformers	1,000
Repairs of meters and operation	1,000
Installation expenses	2,700
Municipal street incandescent repairs	1,000
Commercial administration	5,000
Officers and clerks	3,400
Office supplies and expenses	1,000
	$33,740

The fixed charges for the five years are given in the following table:

Item	Total value				
	1st yr.	2nd yr.	3rd yr.	4th yr.	5th yr.
K.W. hrs. sold	450,000	750,000	1,856,000	3,100,000	4,680,000
Total investment	204,830	209,480	234,010	236,910	268,250
General amortization 5%	10,241	10,474	11,700	11,845	13,412
Insurance) Taxes)2%	4,000	4,200	4,600	4,800	5,360
Total fixed charges per year	14,241	14,674	16,300	16,645	18,772
Operating expense per year	17,585	19,360	24,200	28,900	33,740
Interest at 6% per year	12,289	18,569	14,040	14,214	16,095
Total cost	44,115	52,603	54,540	59,759	68,607

At the end of the fifth year the deficit of the first years is to have been covered and dividends are then to be paid. The total K. W. hrs. sold during the five years is approximately 10,336,000 and the total expenditure $279,624; the rate then would be:

$$\frac{27,962,400}{10,336,000} = 2.7 \text{ cents per K. W. hr.}$$

The ratio, expense to gross income in per cent would be:

$$\frac{68,607}{126,000} = 54.5 \text{ after the fifth year.}$$

The gross income per \$100 invested capital:

$$\frac{126,000 \; . \; 100}{268,250} = \$47$$

Earnings per \$100 invested capital:

$$\frac{57,400 \; . \; 100}{268,250} = \$21.30$$

Altho this figure is somewhat above the average it is due to the fact that the cost of operation of the hydro-electric station is very small.

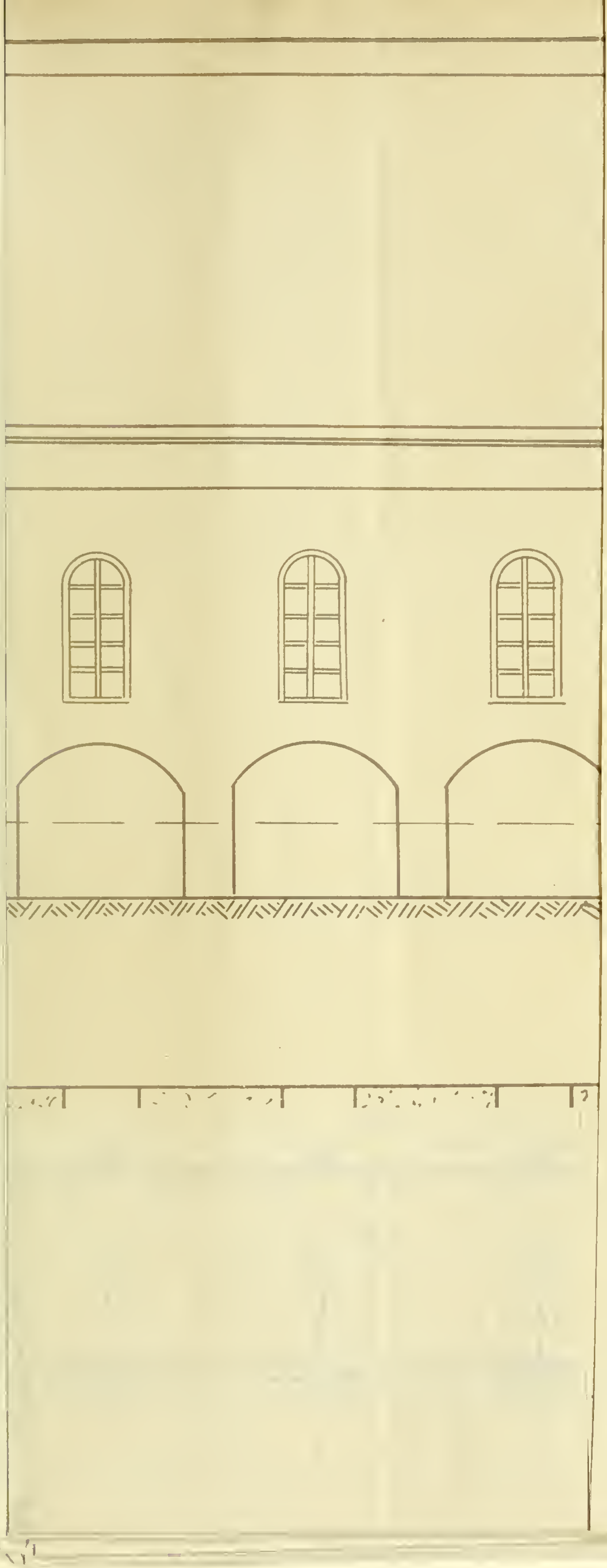


Fig. 10.

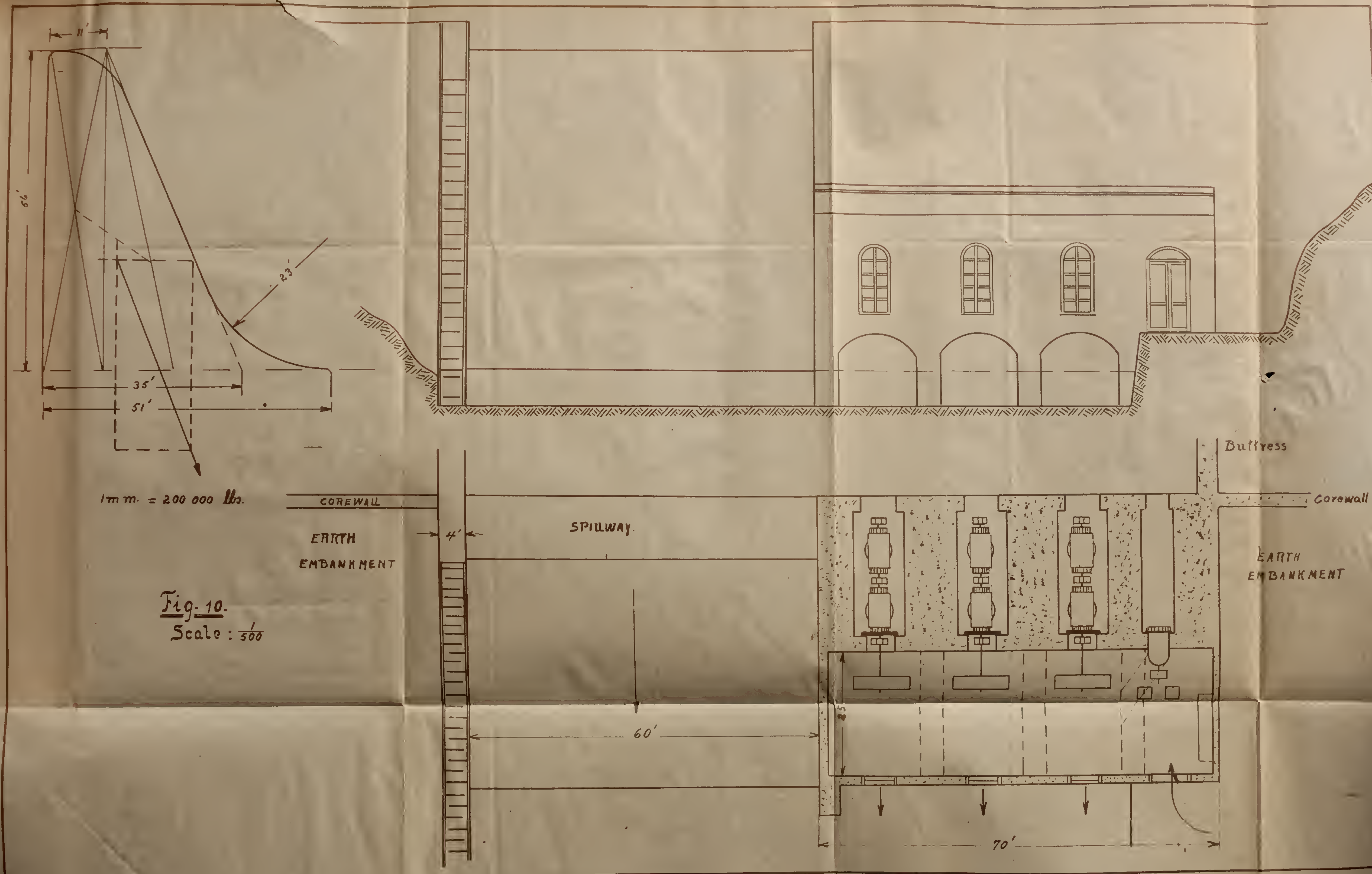


Fig. 10.
Scale : 1/500

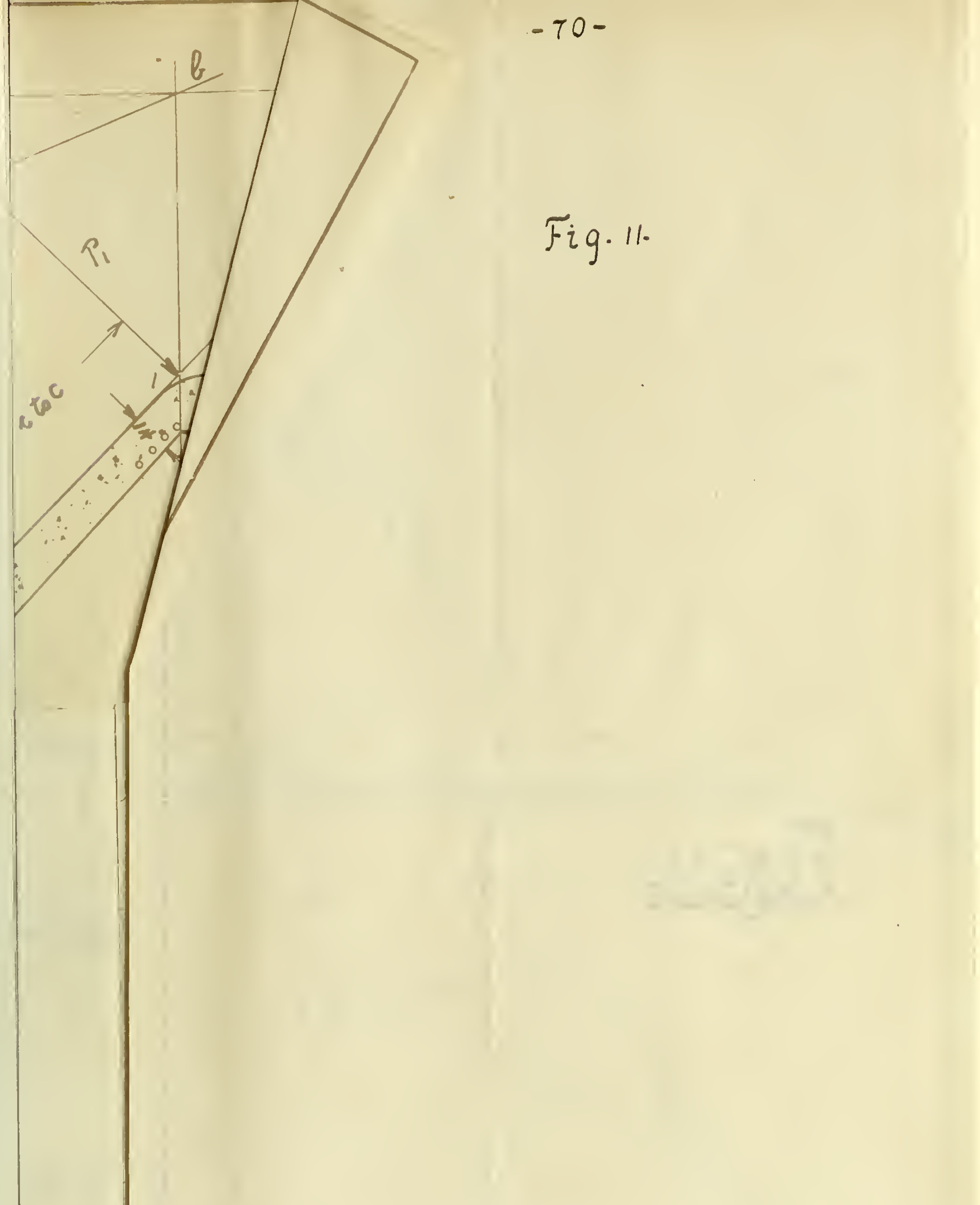


Fig. 11.

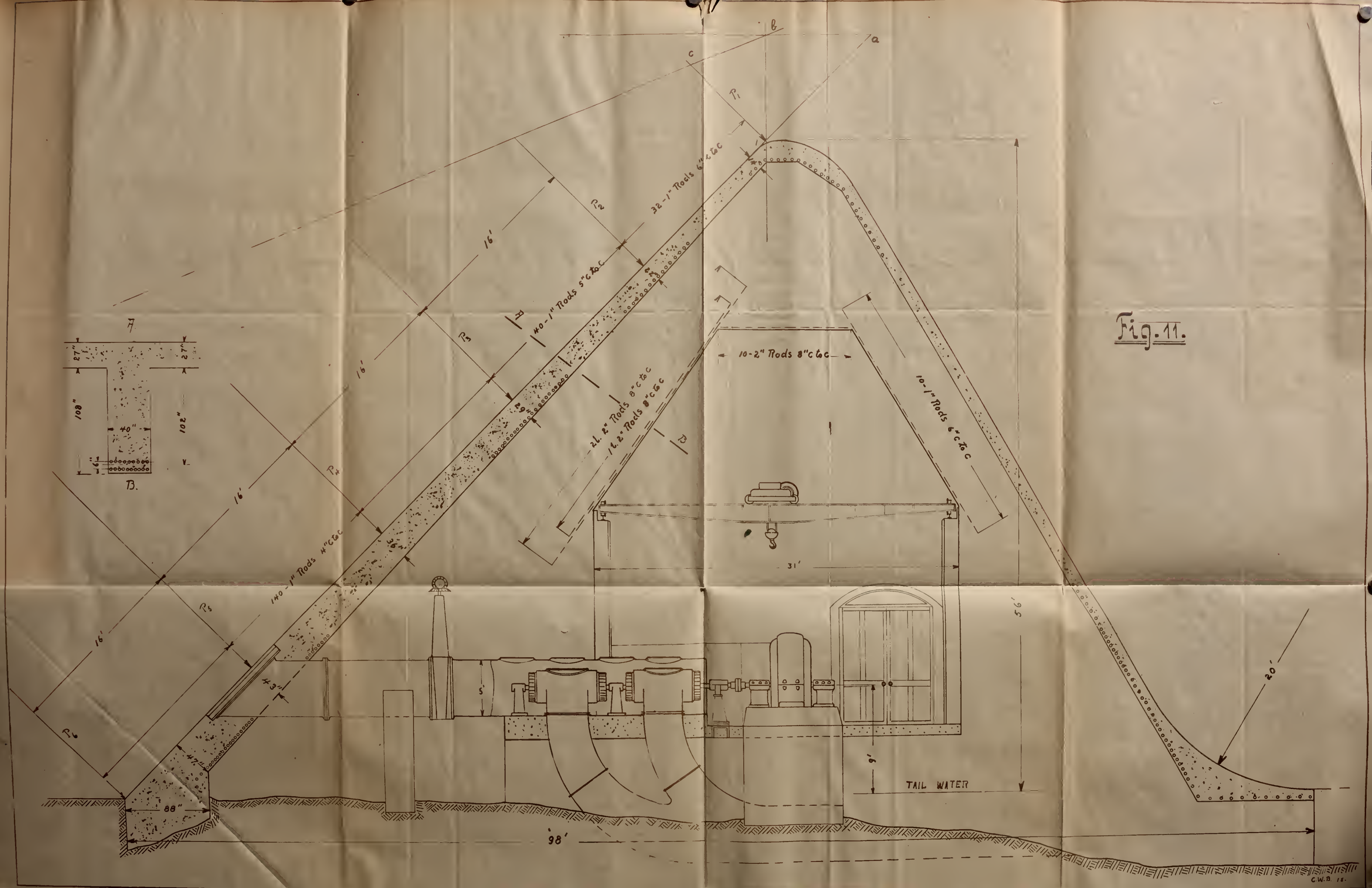


Fig. 11.
A
27"
27"
108"
102"
40"
B.
a
b
c
P_1
P_2
P_3
P_4
P_5
P_6
16'
16'
16'
16'
32-1" Rods 6" c to c
40-1" Rods 5" c to c
140-1" Rods 4" c to c
29"
36"
43"
47"
88"
10-2" Rods 8" c to c
21-2" Rods 8" c to c
14-2" Rods 8" c to c
10-1" Rods 6" c to c
31'
56'
5'
9'
20'
98'
TAIL WATER
C.W.B.

Plate 1

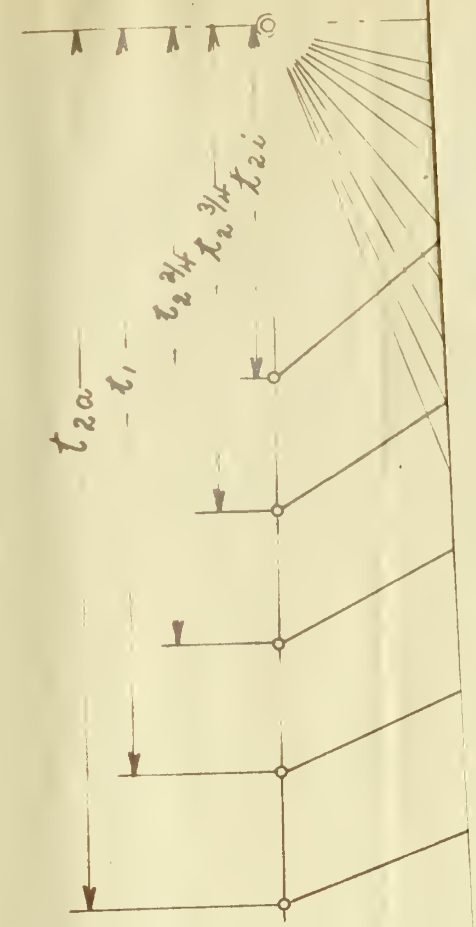


Fig 3.

Plate 1.

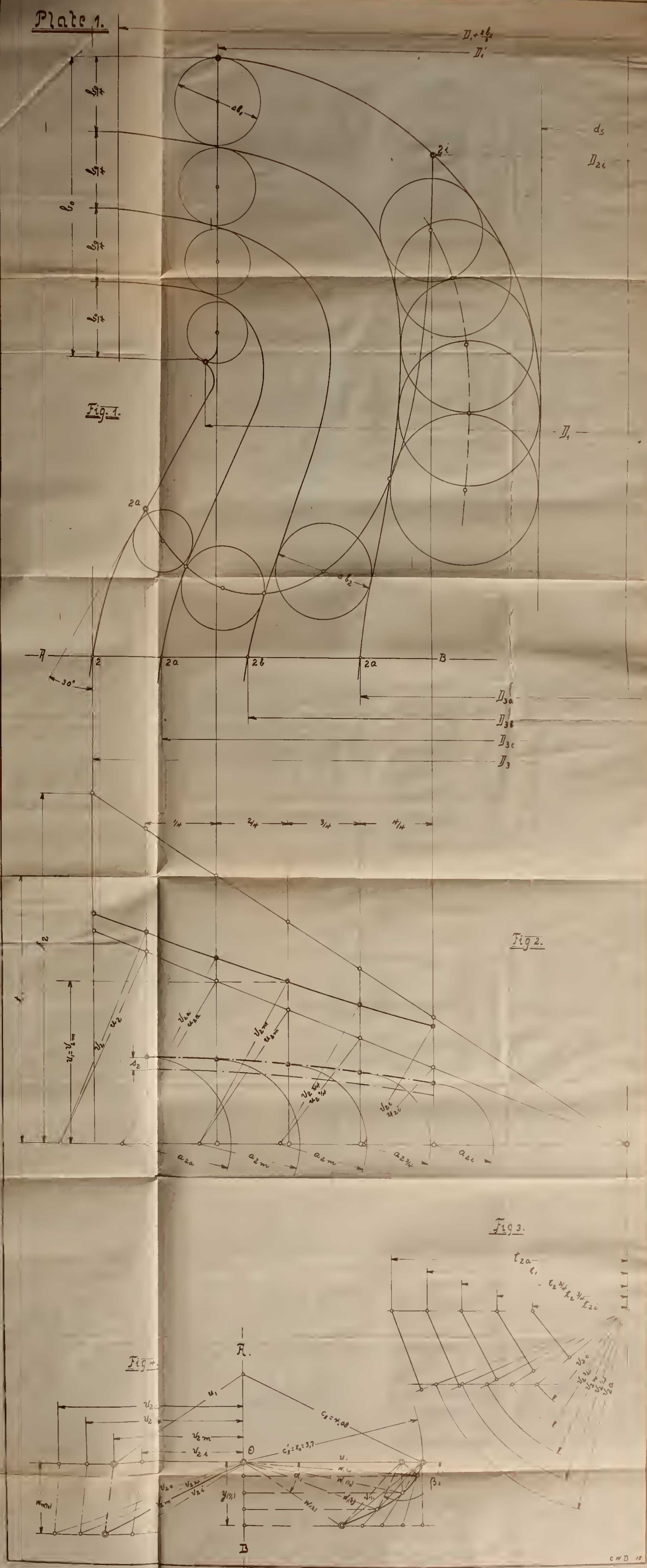


Fig. 1.
Fig. 2.
Fig. 3.
Fig. 4.
A
B
2
2a
2b
2i
30°
D_3
D_{3a}
D_{3b}
D_{3c}
D_1
D_{2i}
d_s
1/4
2/4
3/4
4/4
a_{2a}
a_{2m}
a_{2i}
β_2
u_1
u_2
v_{2a}
v_{2m}
v_{2i}
$c_s = 4.08$
$c_s' = 3.7$
$w_{(1)}$
0
t_{2a}
t_1
t_{2i}
c.w.B. 15

Plate 2

72.

Plate 2.

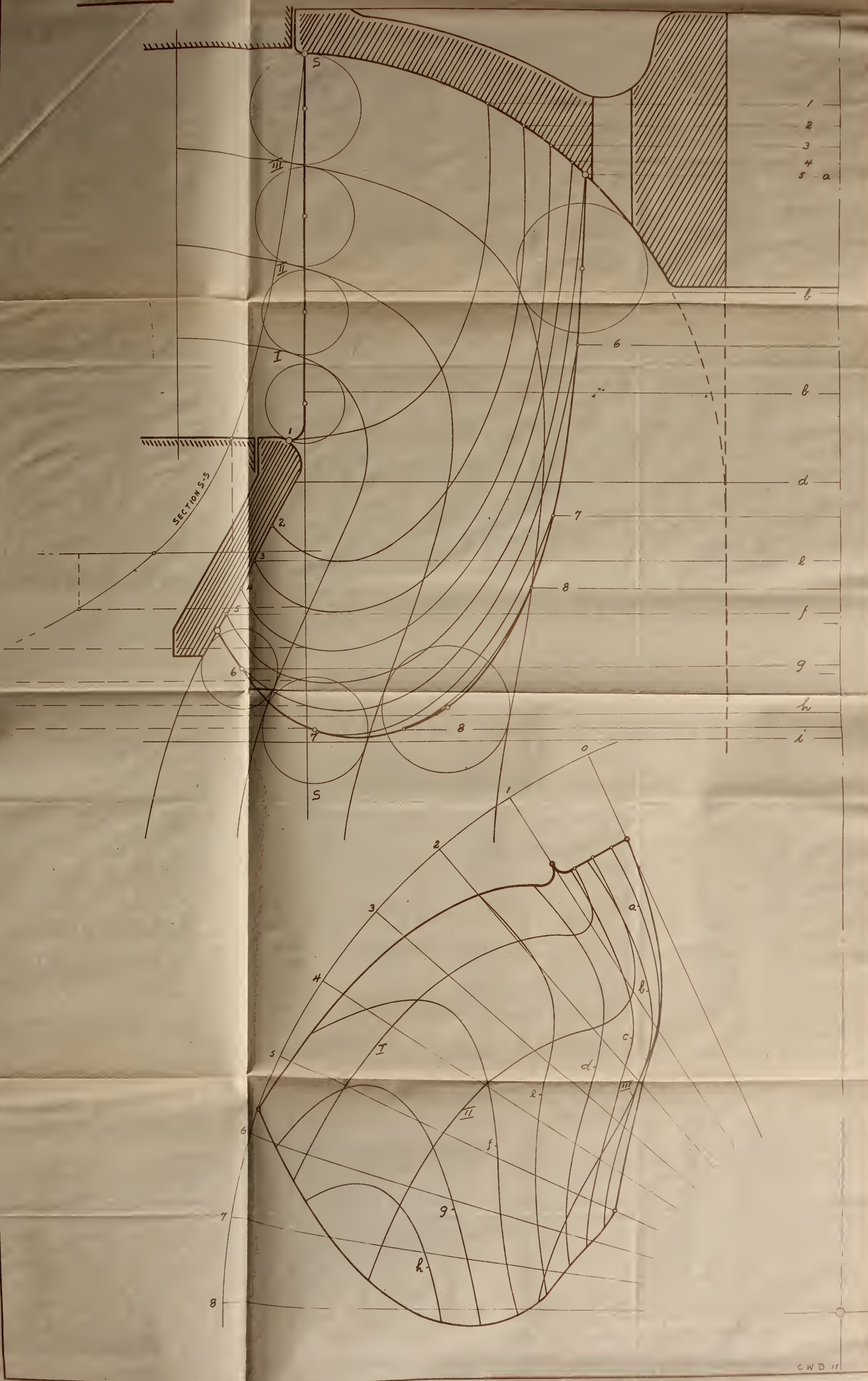
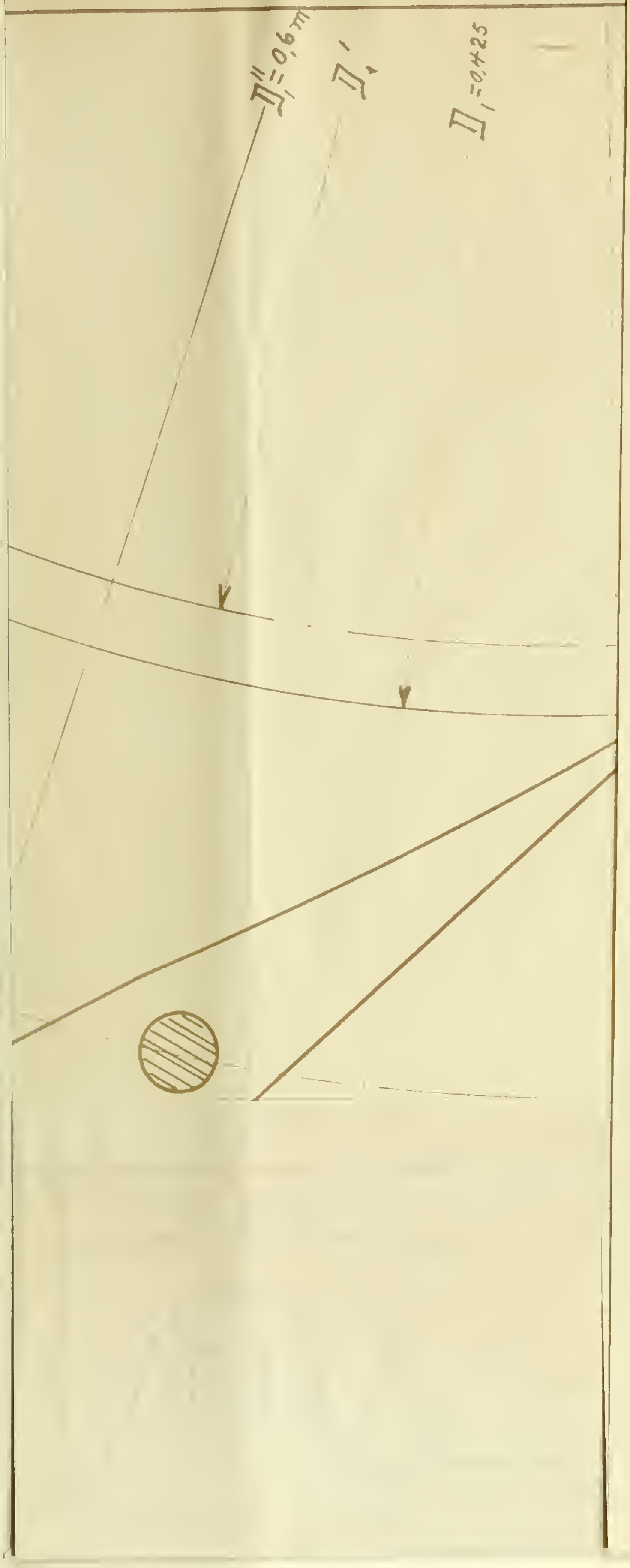


Plate 3.



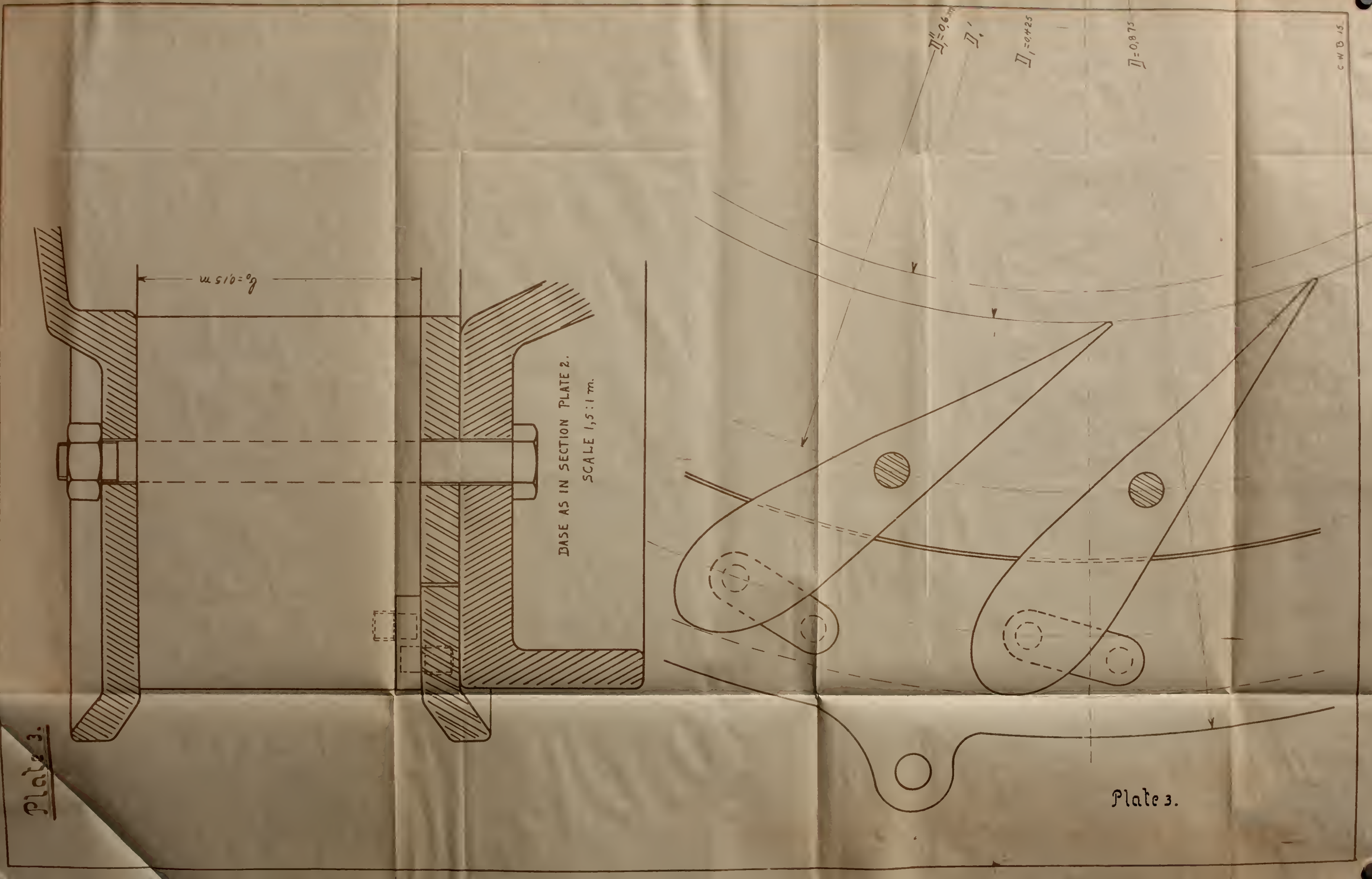


Plate 3.

UNIVERSITY OF ILLINOIS-URBANA
3 0112 086830459